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250

Engineering News

VOLUME 76 AUGUST 3, 1916 NUMBER 5

Hydrolytic Sewage Tanks at Luton, England

By J. W. Tomlinson*

Luton, with a population of 60,000, is an industrial town situated some 30 mi. due north of London. The staple industry is the manufacture of straw and felt hats. There are also a number of industrial engineering works.

Some six or seven years ago it became evident to the authorities of Luton that the existing means of dealing with sewage of the borough were inadequate and unsatisfactory, and steps were taken to put this municipal service upon a better footing. After an investigation extending over a period of two years it was decided to adopt a scheme, prepared by the Borough Engineer, that included tank settlement and double bacterial filtration followed by exposure in large shallow lagoons before discharge to the river.

The consideration given to the question was particularly thorough and careful in view of the fact that the only possible discharge for the final effluent was into the River Lea, from which river, some 30 mi. below Luton, a large quantity of water is taken for the supply of drinking water to the northern parts of London.

So far as the filter beds are concerned, there is but little to distinguish the works at Luton from a large number of others; the shallow lagoons for the exposure of the effluent immediately before discharge into the river are, perhaps, an unusual feature and have justified themselves in quite a marked degree in working, but the settling tanks adopted are of unusual design.

At the present time five of these tanks have been constructed—three for the primary settlement of the sewage as it is delivered by the pumps after no further treatment than passing through a ½-in. screen, and two for the settlement of the effluent from the primary set of filter beds so as to eliminate all solid matters before it passes onto the secondary set of filter beds.

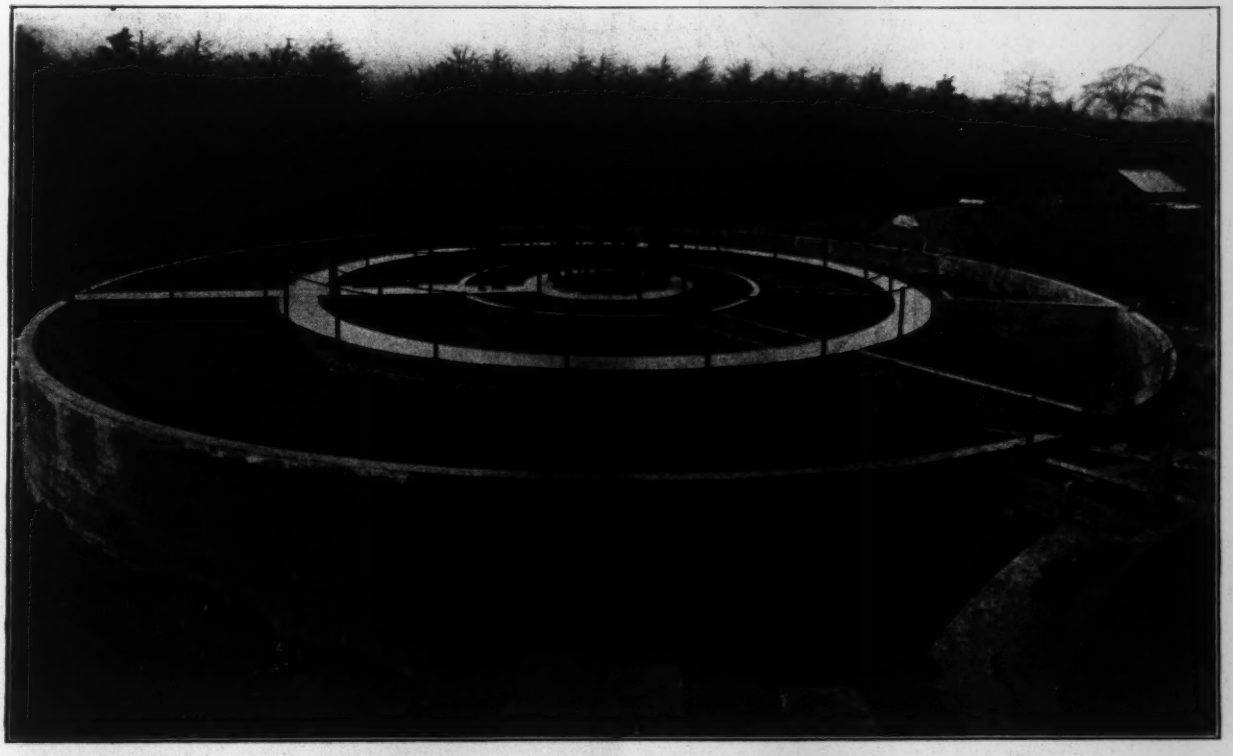


FIG. 1. HYDROLYTIC SEWAGE-TREATMENT TANK, BOROUGH OF LUTON, ENGLAND

The tanks for the two varieties of duty are not identical in design, the secondary tanks being smaller and simpler in type, but the main features are similar

*Borough Engineer and Surveyor, Luton, England.

and it will be sufficient if one of the larger tanks is described in detail.

The governing object of design was to separate as quickly as possible the sludge deposited by the sewage from the main flow of the liquid so as to carry out the settlement continuously without septic action being set up.

The tank (Figs. 1 and 2) has an outside diameter of 65 ft. and a maximum depth of 27 ft. 5 in. At the center is a sludge well, 9 ft. in diameter, which contains no sewage during working, extending below the other compartments of the tank, into which, by means of valves, it is possible to discharge sludge from any chamber. Around this central sludge well is a series of nine chambers arranged in three annular rings and divided from one another by radial and annular walls. The chamber adjoining the sludge wall is only 5 ft. in radial width, and extends for the full circle around the sludge well. The two rings of chambers outside this are separated by a circular wall that is vertical for a distance of 9 ft. down from the top and then inclines outward at an angle of 45°. At the point where this wall reaches the outside wall of the tank there are a series of openings through which the sewage can pass from the outer chamber to the inner one. These chambers are of unequal size, measured circumferentially, the one into which the sewage first enters being smallest and the size increasing so that the last chamber is more than twice the size of the first one. The radial division walls are for the most part hollow and form channels or trunks through which the sewage flows from the surface of one chamber to a point near the bottom of the next.

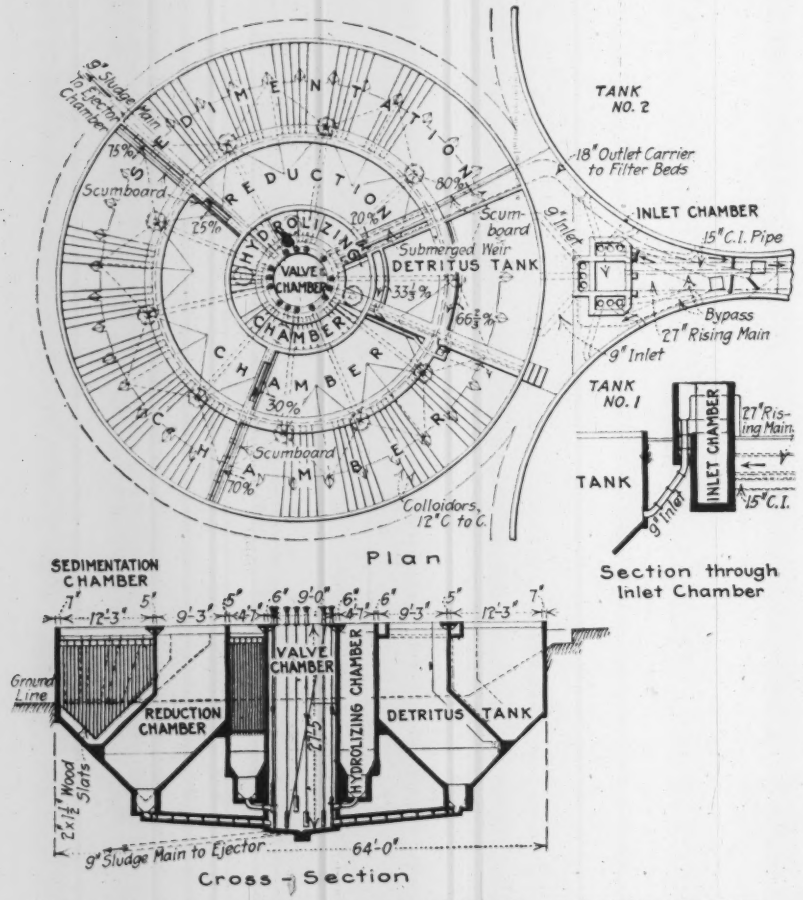


FIG. 2. PLAN AND SECTION OF HYDROLYTIC TANK, LUTON, ENGLAND

The sewage enters the smallest of the outer chambers of the tank some 10 ft. below water level. From this chamber a proportion of the flow passes over a weir, down a trunk in the adjoining radial wall, and is delivered well under water level into the next outer chamber. The remainder of the flow passes through the openings at the bottom of the circular division wall into the inner chamber and subsequently overflowing a weir on the wall of that chamber it passes down a trunk in the radial wall, unites with the overflow from the outer chamber and is discharged into the second outer chamber. In this second chamber the flow is similarly divided, part passing directly forward and part passing, as before, under the dividing wall into the inner chamber. Also, as before, the overflows from both inner and outer chambers pass along trunks in the next radial wall, uniting once more and being delivered into the next outer chamber. The same thing happens in the third chamber, while in the fourth and last of the series, part of the flow passes directly from the outer chamber to the effluent carrier, while the portion from the inner chamber passes into and around the central annular chamber, joining the sludge well before it leaves the tank. The result of this repeated division of flow is that all solids deposited in the outer chambers are carried promptly forward by the flow into the inner chambers, which are deeper, and in the floors of which are situated, at convenient distances, sumps for receiving the sludge. The proportion of flow passing from the outer to the inner chambers is regulated by the comparative lengths of the weirs forming the outlet from the two chambers and is varied to suit the probable quantity of sludge that will be deposited. Thus in the first chamber one-third of the flow passes to the inner chamber; in the second chamber 30%; in the third 25%, and in the fourth 20%.

The floor of the tank is arranged on a ridge-and-furrow principle carried out so completely that there is no portion of it that does not incline at an angle of 45° at least with the horizontal. In consequence of this method of flooring, the sludge deposit quickly finds its way into the several sludge sumps in the floor of the inner chambers.

These sludge sumps are connected by 9- or 6-in. pipes with the sludge well into which the sludge is forcibly discharged when one of the valves in the sludge well is opened.

To aid the collection and settlement of flocculent matter, vertical wooden grids or colloidors (see Figs. 1, 2 and 3) are fixed in the second, third and fourth outer chambers and in the chamber next the sludge well.

These grids are formed of 2x1½-in. slats suspended from a joist crossing the chamber in a radial direction. The suspended matter strikes these rods and forms a slimy deposit on them that in a short time slips off in pieces large enough to settle readily. The grids are wholly under water. They are fixed in groups of five, 12 in. c. to c., and the groups are spaced about 3 ft. apart.

In each compartment a wooden scum board is fixed in front of the overflow weir.

The sludge that is drawn off into the sludge well from the sludge sumps in the different chambers of the tank passes from thence by gravitation along a 9-in. outlet pipe to a 100-gal. (Imp.) automatic ejector, operated by compressed air, by means of which it is raised onto the surface of the land at some distance from the tanks, where it is dealt with. The air compressor is motor driven.

There is thus no need to empty the tank after it commences work and its operation is continuous. As a matter of fact, three of the tanks at Luton have been in operation for five years, during which time they have never been emptied and they work quite as efficiently now as ever.

Each of the primary tanks at Luton has a capacity of 260,000 gal. (Imp.), and two such tanks deal quite efficiently with a daily flow of 1,500,000 gal. (Imp.).

The tanks are constructed of reinforced concrete throughout. The outer wall is 7 in. in thickness, the wall round the central sludge well 6 in. thick and the other walls mainly 5 in. The reinforcement is square indented steel bars, the sizes used being ⅛, ½, ⅝ and ¾ in. The vertical reinforcement is of ⅛-in. bars throughout, about 18 in. c. to c. The horizontal reinforcement, 6 to 9 in. c. to c., varies in size from ⅛ to ¾ in. according to depth, size of circle and weight to be carried. The concrete used was mixed in the proportion of 6 to 1, the cement being of British standard quality and the aggregate consisting of 50% ⅝-in. Leicestershire granite chips, 25% ¼-in. chips of the same material and 25% of coarse local sand.

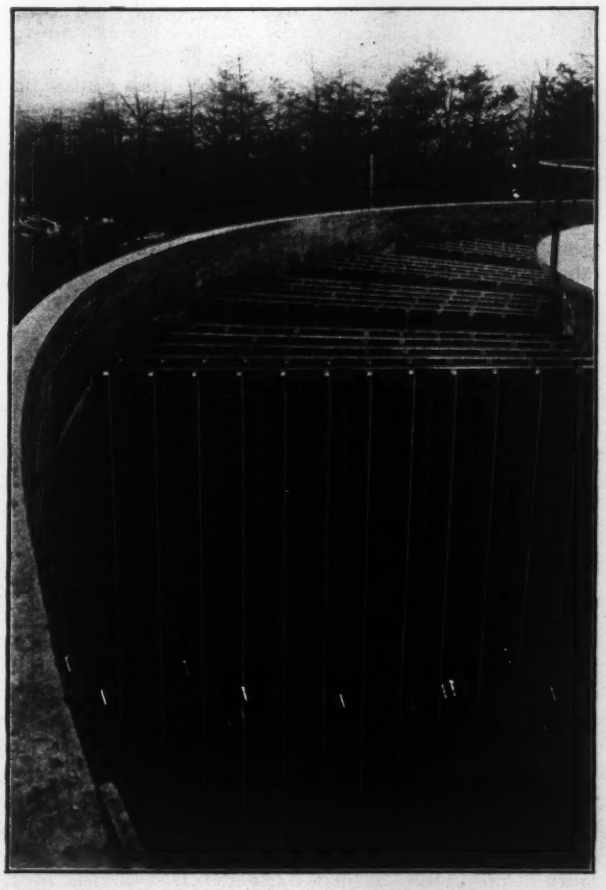


FIG. 3. WOODEN COLLOIDOR GRIDS IN LUTON TANK

The earth was excavated to the configuration of the floor of the tank and the 6-in. concrete floor laid. The outside and two innermost circular walls were then constructed, the same forms being used several times over; the radial walls were next formed and finally the shaped circular wall dividing the outer and inner chambers. When the boarding was removed any projecting ridges on the concrete were dressed off, the surface brushed over with a wash of neat cement and then plastered.

The workmen employed were not skilled. None of them had had any previous experience with reinforced-concrete work and yet no trouble was experienced in obtaining a thoroughly sound job, quite water-tight at the somewhat considerable pressure that occurs.

The efficiency of the design may be gathered from the following analysis of representative sewage and tank effluent, the samples being obtained by adding together equal quantities taken each hour for 24 consecutive hours.

	Parts per 100,000					
	Solids			Nitrogen		Oxygen
	In Suspension	In Solution	Chlorine	Ammoniac	Albumin	Absorbed in 4 Hr.
Crude sewage...	57	117.4	14.6	5.75	0.95	8.1
Tank effluent...	3	111	13.5	5.8	0.28	4.9

This shows a reduction of 94% in the suspended solids and of 70% in the albuminoid nitrogen—a very good result.

The crude sewage is generally colored by dye waste, often to a vivid degree, and the tank effluent in consequence is usually of a somewhat yellowish tint.

The cost of operating the tanks is very small, being confined to the cost of the daily removal of the sludge. With electrical current available at 1d. per unit the running costs, including man's time, are about 8 shillings ($2) per week per tank.

The original cost of the tanks is very small compared with the flow of sewage they will treat. Thus two primary tanks actually cost £3,405 or $16,550 (including patentee's fees) and deal with a flow of 1,500,000 Imp. gal. or 1,800,000 U. S. gal., which is equal to a cost of £2,270 per 1,000,000 Imp. gal., or about $9,200 per 1,000,000 gal. per 24 hr.

The sludge-lifting plant, comprising electrically driven air compressor, pipe lines, two automatic ejectors, chambers, engine house, etc., cost £1,409, or $6,850, and this plant is capable of dealing with double the present number of tanks.

The design of the tank has been patented and the holder of the patent is the Hydrolytic Tank Construction Co., 16 Victoria St., London, S. W., England.

An Irrigation System for the Reeves County Irrigation District No. 1, Balmorhea, Tex., is to be built. Bids will be received June 24 for a diversion dam, two earth storage dams, and canals. There will be about 180,000 cu.yd. of excavation, 185,000 cu.yd. of embankment, 22,000 sq.yd. of dry paving and 4,000 cu.yd. of concrete. V. L. Sullivan in engineer and W. W. Stewart is Secretary of the district.

Tunnel Grouting in Brooklyn End of Catskill Aqueduct

By O'Kelly W. Myers*

SYNOPSIS—Methods of controlling water inflow while placing concrete lining and of grouting to stop subsequent leaks are described. Sequence of placing concrete lining is given; also method of controlling water by pans and piping and at worst place by steel lining to convey water to a drainage sump. Cost figures are presented.

The concrete lining of that portion of the city tunnel between shafts 23 and 24, a part of the Catskill Aqueduct located in Brooklyn, is of interest because of the amount of water inflow through the rock. It is believed that the

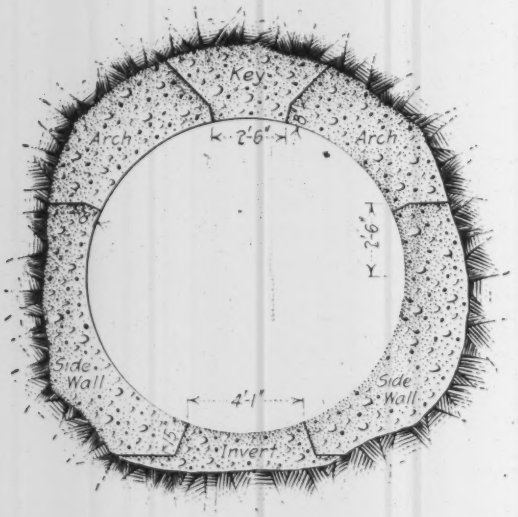


FIG. 1. SKETCH SHOWING CONSTRUCTION JOINTS IN CONCRETE LINING OF TUNNEL

methods used and the results obtained in handling the water while the concrete was being placed and in closing off this water by grouting are worthy of notice.

The portion of tunnel considered here is 3,600 ft. long and 11 ft. finished diameter. The rock encountered in sinking the shafts and in driving the tunnels is known as granodiorite, being a somewhat similar formation to the gneiss. In shaft 23 and for the first 600 ft. of tunnel leading therefrom the rock was sound and dry, but in shaft 24 and the remainder of the tunnel frequent seams and crushed areas were encountered. From these seams and crushed areas water flowed, the leaks being numerous and varying from very slight seepage to a maximum of 240 g.p.m. (gallons per minute) from a seam in shaft 24 and a maximum of 600 g.p.m. from one in the tunnel near shaft 23.

The elevation of the finished invert of the tunnel is —269, the surface of the overlying rock varying from —73 to —105 and the ground water level —5. The water flowing into the tunnel was therefore under a head of 264 ft., or about 114 lb. pressure per square inch.

*Assistant Engineer, Board of Water-Supply, New York City.

The maximum leakage recorded for the tunnels during the driving was 600 g.p.m., this coming from one seam. The leakage from the entire tunnel, upon completion of driving, amounted to only 475 g.p.m., although numerous large and small leaks were encountered, the reduction being due to silting; the 600-g.p.m. flow having reduced to about 300 g.p.m. is an example. This leakage was further reduced to 440 g.p.m. when the concrete lining had been completed.

The tunnels are lined with 1:1½:3 concrete, which has a minimum thickness of 15 in. with an average thickness of 2 ft. The lining was placed in four steps (Fig 1), as follows: The invert, 4 ft. wide, was laid in stretches of about 200 ft.; the sidewalls were placed in 60-ft. stretches; the arch was placed in 60-ft. stretches; and the key, at the back end of the form, was a continuation of placing the arch concrete. This key was generally completed 6 hr. after finishing of the third step.

In the longitudinal joints no means were taken to insure tightness other than removing the laitance and finishing the surface, when forms were not used, with a wooden float or cement bag. In the transverse joints a key block made of short pieces of 4x4-in. timbers, beveled, was placed 8 in. or more from the face. It is doubtful if the key formed by these blocks has proved to be of any benefit.

Upon the removal of the forms transverse contraction cracks developed, there being one or two in each 60-ft. section. These cracks generally encircled the tunnel, including the invert.

The lining, so far as the body of the concrete was concerned, was tight; but there were numerous small leaks and seepage from the key joints, the transverse joints and particularly from the contraction cracks.

The handling of the water in shaft 24 was done in a somewhat different way and at a different time from that in the tunnel, so will be considered separately. Little trouble was encountered in handling the water in the shaft, except in one stretch of 40 ft., where one leak gave a flow of 240 g.p.m.

The shaft, which is circular and varies from 16 to 18 ft. finished diameter, was sunk, lined and grouted in

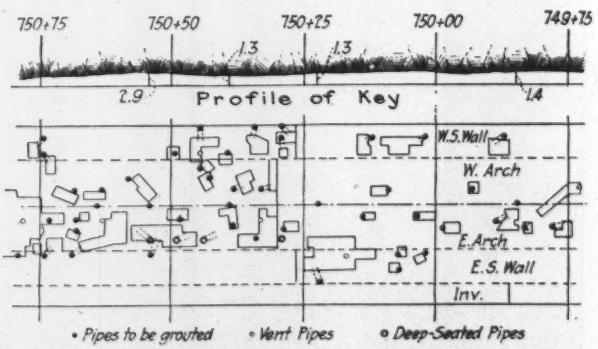


FIG. 2. LOCATION OF DRIP PANS AND INSPECTION PIPES SHOWN BY DEVELOPMENT OF TUNNEL LINING

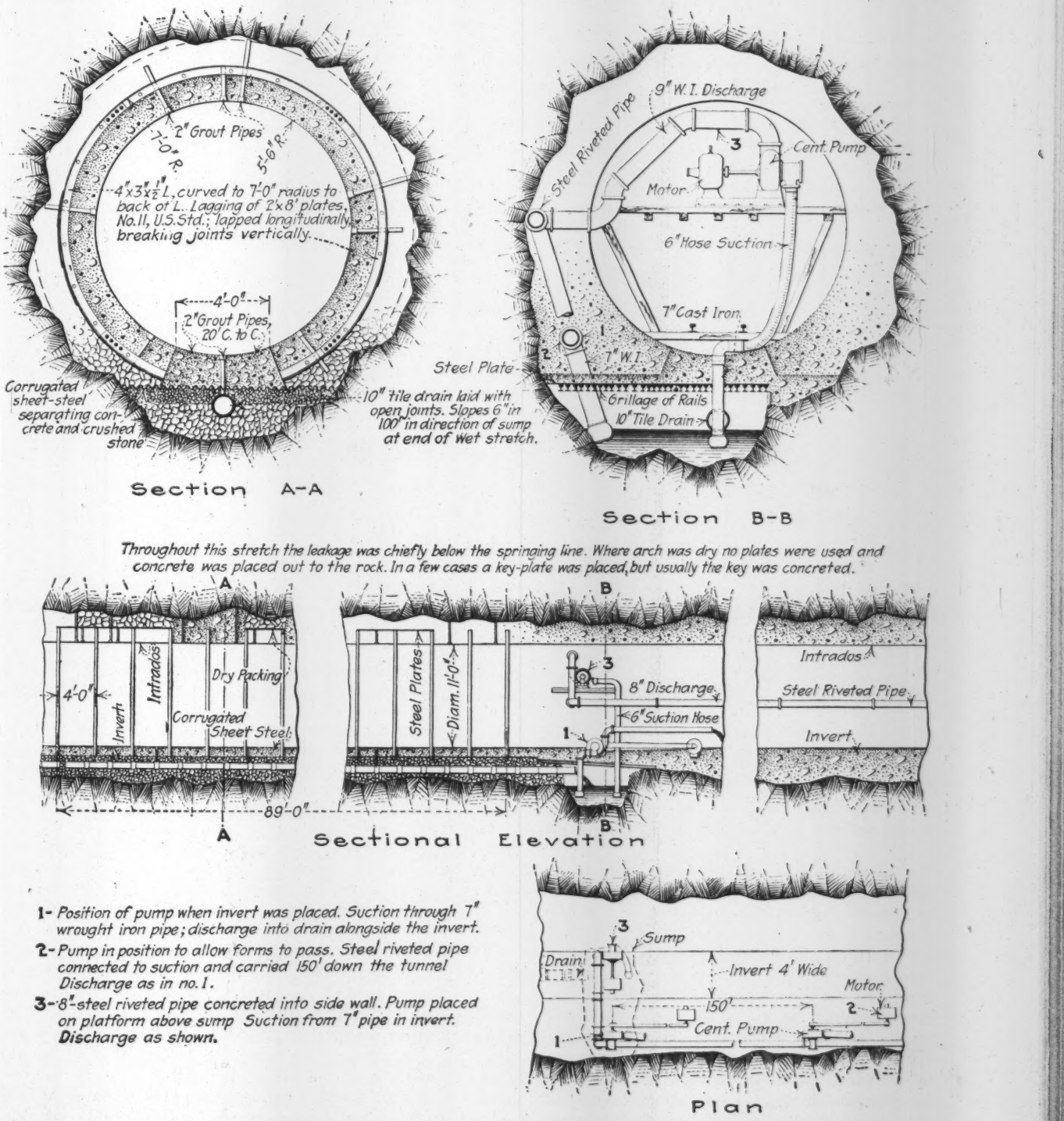

2" Grout Pipes
7'-0" R.
5'-6" R.
4"x3"x1/2" L., curved to 7'-0" radius to back of L. Lagging of 2"x8" plates, No. 11, U.S.Std., lapped longitudinally, breaking joints vertically.
4'-0"
2" Grout Pipes, 20' C. to C.
Corrugated sheet-steel separating concrete and crushed stone
Section A-A
9" W.I. Discharge
Steel Riveted Pipe
Cent. Pump
Motor
6" Hose Suction
7" Cast Iron
7" W.I.
Steel Plate
10" tile drain laid with open joints. Slopes 6" in 100' in direction of sump at end of wet stretch.
Grillage of Rails
10" Tile Drain
Section B-B
Throughout this stretch the leakage was chiefly below the springing line. Where arch was dry no plates were used and concrete was placed out to the rock. In a few cases a key-plate was placed, but usually the key was concreted.
A
4'-0"
Invert
Intrados
Dry Packing
Corrugated Sheet Steel
89'-0"
Steel Plates
Diam. 11'-0"
8" Discharge
6" Suction Hose
Intrados
Steel Riveted Pipe
Invert
Sectional Elevation
1- Position of pump when invert was placed. Suction through 7" wrought iron pipe; discharge into drain alongside the invert.
2- Pump in position to allow forms to pass. Steel riveted pipe connected to suction and carried 150' down the tunnel Discharge as in no. 1.
3- 8"-steel riveted pipe concreted into side wall. Pump placed on platform above sump Suction from 7" pipe in invert. Discharge as shown.
Sump
Drain
Invert 4' Wide
Motor
150'
Cent. Pump
Plan

FIG. 3. CORRUGATED STEEL LINING AND ARRANGEMENT OF PLANT TO CONTROL WATER NEAR SHAFT 23

stretches not exceeding 100 ft. During the sinking numerous small seams were encountered which flowed water, but became dry as seams below were reached by the excavation, until at El. —221 a seam which evidently fed those above was encountered. The water from this seam flooded the shaft, requiring the installation of a large pumping plant to reclaim it. This water had been reached by a drill hole passing through about 5 ft. of sound rock. When the shaft had been emptied, this hole was fitted with a pipe and valve, and a series of holes encircling the shaft was drilled through to the water. Through pipes fitted in these holes 57 cu.yd. of liquid grout was forced. The grouting of this seam caused the water to rise in the rock and flow from smaller seams above, so while waiting for this grout to harden an effort was made to close the seams above. This was done by drilling holes in the seams, into which grout under low pressure was forced, wedges of soft wood being driven into the seams where necessary to prevent the grout from escaping into the shaft. Two cubic yards of

grout was used, reducing the leakage from 50 to 15 g.p.m.

Upon resuming excavation it was found that the seam which gave the large flow had been successfully grouted and gave no trouble; but after the seams above were closed, the rock scaled so badly that it was necessary to suspend sinking and place the lining. The water remaining was handled through the concrete and later grouted by the same methods as described for the tunnel.

In handling the water in the tunnel, wherever possible any flow of 2 or 3 g.p.m. or more was collected into a deep-seated pipe and led through an opening in the form. Where smaller flows or general seepage was encountered and in connection with the deep-seated pipes, drip pans were used to collect the water, which was led from the pans through the forms by pipes (Fig. 2).

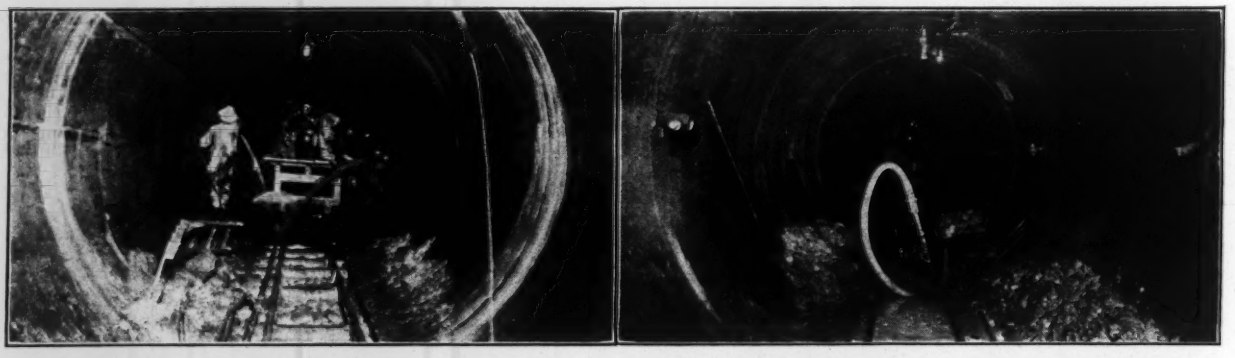
The pans consisted of sheets of No. 24-gage black iron cut to the desired size. The area to be panned was outlined, and 1-in. holes were drilled about 15 in. apart into the rock, the holes being 2 in. deep. Wooden plugs were driven into these flush with the rock face. The sheet iron was nailed to the plugs, the edges calked with oakum and plastered with cement or plaster of paris. The precautions of calking and plastering were taken to prevent the mortar from the concrete from getting behind the pan. The pans varied in size from one of 4 or 5 sq.ft. to a maximum of 150 sq.ft. Where possible, several small pans rather than one large one were used, thereby lessening the liability of damaging the concrete when placing the high-pressure grout. The pans were generally fitted with two pipes—one at the bottom to handle the water and later to take the grout, the other near the top as a vent to aid the circulation of the grout in the pan. A few small pans were connected to only one pipe, and in one instance a long narrow pan, following a seam, had 10 pipes. The pans were generally made to conform with the rock surface; but in some instances, in order to protect the metal from sharp edges of rock, or when a large amount of water was to be handled, they were partly filled with crushed stone. Much attention was given to the pans, and this care was justified when it is considered that the plugging of a pipe may mean the escape of the water into the green concrete and the possible removal and replacing of several cubic yards of porous concrete.

In the roof, to take care of the shrinkage of the concrete, key pipes were placed, generally about 10 ft. from each end of the 60-ft. section, the location selected being alternate places of low and high breakage. An unusual roof breakage was taken care of with a separate pipe.

All pipes were 2 in. in diameter and, when possible, straight, this being desirable as it is frequently necessary to clean out a pipe while grouting. In the 3,600 ft. of tunnel there were placed 215 pans and 781 pipes.

One section of the tunnel near shaft 23 (Sta. 5,761 + 00 to Sta. 5,761 + 89) flowed a large amount of water, 600 g.p.m., when first encountered. It was therefore decided to use a special construction rather than take the risk of damaging the green concrete, should drip pans be relied upon. This construction is shown by Fig. 3.

In this case the steel lining may be considered as a large drip pan attached to the metal ribs instead of to the rock, the water being led down to the drain and the sump instead of being carried through the forms and the plates, and dry packing being placed only where there was leakage to be handled, over about 60% of the area. The steel shell lining was extremely light, being designed neither to carry any strains nor to act as reinforcement.



FIGS. 4 AND 5. GROUTING CARRIAGE AS SEEN FROM STATION 5761 + 50, NEAR SHAFT 23, BROOKLYN SECTION OF CITY TUNNEL, CATSKILL AQUEDUCT

Fig. 4—Looking north. Near view of carriage and water flowing from deep-seated grout pipes. Low-pressure grouting completed. Note water discharging from pipes. Fig. 5—Looking south. Front of car in distance and connection to invert pipe in foreground, showing method of cleaning high-pressure connections. High-pressure grouting completed

The plant used in grouting consisted of two Canniff machines connected in series and two air compressors capable of raising the pressure from 100 to 350 lb. per square inch, placed on a small flat-car with a trailer, upon which the material was stored and the machines charged. Air was furnished through a 4-in. pipe line at 90-lb. pressure. A 2-in. line connected to the discharge pipe of the tunnel pumps furnished the water. Fig. 4 shows the grouting carriage near-by, and Fig. 5 farther removed, at two different stages of the work.

Beginning in the tunnel at the foot of shaft 24, grout consisting of one bag of cement and an equal volume of beach sand with 8 gal. of water (this being equal to 2½ cu.ft. of liquid grout or 1.66 cu.ft. of solid) was forced into the key pipes under pressure not exceeding 90 lb. until the pipe refused to take more or until the grout had moved along the tunnel for 250 or 300 ft., all of the valves on the key pipes and a number of the pan pipes above the springing line having beeen closed as grout began to flow from them.

This step, called the low-pressure grouting, was intended to fill the void between the concrete and the rock surface of the roof caused by the shrinkage and settlement of the concrete, as well as to confine the grout placed later under high pressure. As the tendency of the

grout is to settle in the low places, leaving voids where the roof is high, the long pipes were cleaned out, after the grout had begun to stiffen and before it had set, for future connections.

Sixteen hours after the low-pressure grout had been placed, all valves were opened and any pipes showing leakage or any deep-seated pipes not previously opened were drilled out for high-pressure connection. The low-pressure grouting was a continuous operation, a three-shift organization being used, and was completed before any high-pressure work was started. The grout circulated along the roof so freely that connections to the key pipes, for the low-pressure work, were necessary only at intervals of from 150 to 200 ft. There were placed in this step 4,878 batches, which represents 2.193 cu.ft. of solid grout per linear foot of tunnel.

In the high-pressure work a neat cement grout consisting of one bag of cement to 6 gal. of water, making 1.4 cu.ft. of liquid grout, was used and was forced in under a pressure of 300 lb. per square inch, the pressure being maintained for about 6 min. after the grout had ceased to flow, this being the time found necessary to plug or dry out the grout in the pipe. By using a blowoff near the valve the liability of plugging the hose was removed. Connections were made to the lower pan pipes, all upper pipes being closed upon their venting grout. Later, before the grout in the upper pipes had set, connections were made to them, as it was found in most instances that those pipes would not be entirely filled, due to the presence of the water contained in the grout, the air being unable to drive it away.

The deep-seated pipes (in the seams which had been making water during the low-pressure work) were connected to and were grouted after the local pan pipes had been plugged. Generally only enough grout to fill the void formed by the pan could be forced into the pipes, except the deep-seated pipes and some of those near the roof where the grout had chance to escape to the roof.

In a few instances, due to the tightness of the rock and consequent retention of air and water in the pipes or to small leaks in the valves the pipes were not successfully closed, but upon being cleaned out and a connection maintained from 10 to 15 min. they would be found to be closed upon the removal of the valves.

The greatest difficulty encountered was in closing off the water coming through the contraction cracks above the springing line. These cracks generally flowed grout freely during the low-pressure grouting, but would close, partly opening again, after the grout hardened. As these cracks were not necessarily convenient to the pipe placed for other conditions, there were a few cases where the high-pressure grouting could not reach them and they continued to leak; but when near a key or pan pipe, the leakage was cut off without trouble. A few of the worst of these leaks were selected for experiment. The cracks were drilled into, pipes placed and grouted, the result in each instance being satisfactory. The remaining leaks were so small that the conditions would not justify the time or expense to close them. The leakage from these seams is decreasing through siltage, it is supposed.

The grouting in the stretch in which the special construction was used was not begun until all other grouting had been completed. After a careful study of the special conditions existing, the following operations were carried out: Three key pipes at each end beyond the stretch were left open as vent pipes during the last step of the low-pressure grouting; all pipes below the springing line except the deep-seated pipes were closed; the pipes at and above the springing line were left open; the first connection was made to the pipe at the upper end of the drain, and when this refused grout under 150-lb. pressure the next pipe in the drain was grouted and so on until all drain pipes were filled; then returned to the upper end of the drain, connecting to the pipes leading to the dry packing in the bottom; again returning, connecting to the springing-line pipes; the last step was to connect the key and deep-seated pipes, working in the opposite direction, using high pressure.

In order to keep down the pressure from the outside, all pipes except the invert pipes were left open or opened after the grout had settled. In this way most of the grout was placed against a pressure of 50 lb. or less, the final reading of the gage being 90 lb. when the grouting plugged the gage.

The method outlined gave results far better than had been hoped for. The grout forced into the drain pipe rose above the springing-line pipes without traveling any great distance ahead, a similar action being noticed from the other pipes, the retained water probably acting as a cushion, making it as easy for the grout to rise through the clean dry packing as to flow along the bottom through the packing filled with the solid grout.

The deep-seated pipes and the roof pipes flowed grout and had to be closed when grouting was being done in a near-by pipe, but could be opened later. This was a great aid, as it permitted most of the grout to be placed under low pressures, thus saving time and insuring a better circulation of the grout.

Assuming that the dry packing contained 45% voids, it was estimated that 1,350 batches would be placed under low pressure. There were actually placed 1,420. As expected, the low-pressure grouting did not to any extent reduce the flow of water, but filled the voids, forcing the water to the deep-seated pipes.

It was found upon opening the valves that but few of the pipes below the key had been plugged by the low-pressure work; consequently all pipes were opened and connected to under high pressure, several of the invert pipes requiring a second connection. Altogether, 217 batches were used in the high-pressure work on this section.

This section is now very dry, and that it was ever otherwise would not be noticed except for the number and the regularity in the placing of the pipes. The leakage for the 3,600 ft. of tunnel has been reduced to 6 g.p.m. and probably could be reduced still further, should it be considered necessary to resort to calking; but as it is expected that silting will reduce this leakage, no attempt will be made at present to reduce it by calking.

To complete all of the grouting, 96 shifts of 8 hr. were required. The gangs were green to this class of work and had to be trained. Each gang consisted of one foreman and seven men.

A total of 76 low-pressure and 521 high-pressure connections was made, taking 578 cu.yd. of low-pressure and 121 cu.yd. of high-pressure grout. The record for 8 hr. was 422 batches under low pressure.

The cost of the work is detailed in the accompanying table, shown on the next page.

GROUTING COSTS IN TUNNELS 23-N AND 24-N FROM CONTRACT 67 ESTIMATES

Description	Quantity	Unit Cost	Amount	Total	Cost per Ft.
1,611-Ft. Tunnel 23-N Excepting Wet Stretch					
Drilling 1½ - in. holes or smaller	86 ft.	$1.00	$86.00	.....	....
Drilling 1¾ - in. holes or larger.	38 ft.	2.50	95.00	.....	....
Steel pipe for grouting	2,186 ft.	.50	1,093.00	.....	....
Connecting machines to grout pipes	468 con.	1.50	702.00	.....	....
Sand for grout...	119 tons	2.00	239.00	.....	....
Mixing and placing grout	309 cu.yd.	6.00	1,854.00	.....	....
Portland cement..	976 bbl.	1.50	1,464.00	.....	....
				$5,532.00	$3.43
89-Ft. Tunnel 23-N—Wet Stretch (Sta. 5,761 + 00 to 5,761 + 89)					
Drilling 1½ - in. holes or smaller	.....	1.00	.....	.....	....
Drilling 1¾ - in. holes or larger.	6 ft.	2.50	15.00	.....	....
Steel pipe for grouting	183 ft.	.50	91.50	.....	....
Connecting machines to grout pipes	71 con.	1.50	106.50	.....	....
Sand for grout....	80 tons	2.00	160.00	.....	....
Mixing and placing grout	142 cu.yd.	6.00	852.00	.....	....
Portland cement..	407 bbl.	1.50	610.50	.....	....
				1,835.50	20.62
1,700-Ft. Tunnel 24-N					
Drilling 1½ - in. holes or smaller	22 ft.	1.00	22.00	.....	....
Drilling 1¾ - in. holes or larger.	9 ft.	2.50	22.50	.....	....
Steel pipe for grouting	1,445 ft.	.50	722.50	.....	....
Connecting machines to grout pipes	327 con.	1.50	490.50	.....	....
Sand for grout....	103 tons	2.00	206.00	.....	....
Mixing and placing grout	246 cu.yd.	6.00	1,476.00	.....	....
Portland cement..	767 bbl.	1.50	1,150.50	.....	....
				4,090.00	2.41
Total for 3,400 ft.				$11,457.50	$3.37

The work described is being carried out by the Board of Water-Supply of New York City, of which J. Waldo Smith is chief engineer. W. E. Spear is department engineer, City Aqueduct Tunnel, and James F. Sanborn was division engineer at the close of the work.

Water-Evaporation Studies by Weather Bureau

By Benjamin C. Kadel and Cleveland Abbe, Jr.*

Discussions of a paper recently published in an engineering journal show that there is need of a statement by the Weather Bureau as to its present position relative to the evaporation work heretofore carried out under the supervision of the bureau and also as to the bureau's present plans for the conduct of work of this character.

The results of the extensive campaign, popularly known as the Salton Sea campaign, carried on by the Weather Bureau from 1907 to 1911, were presented in a preliminary way in various numbers of the *Monthly Weather Review* of the same period; but as some questions arose thereafter as to the basis of discussions used at that time, the publication of the full report was deferred until it might be possible to revise all the work thoroughly. The original records are intact and without doubt contain many data of value. In order that this fund of information might be made available the Chief of the Weather Bureau, in February, 1915, appointed Benjamin C. Kadel and Dr. Cleveland Abbe, Jr., a committee to select the most valuable of these data and transcribe them for publication. This work has gone forward as rapidly as possible, and the data, in the form of a transcript of the original observations, will be published in the near future, probably in the form of a supplement to the *Monthly Weather Review*. In addition to this preparation for presenting to the public the information already in the possession of the Bureau a new series of evaporation stations is now being installed at various points throughout the United States. These stations[1] consist of an outfit selected with the object of so standardizing exposures and instruments that a comparison of values of evaporation in various portions of the country may be possible. It should be recognized that the demand for information concerning evaporation proceeds from widely different interests. The engineer wishes to determine evaporation losses from lakes, reservoirs and artificial supplies of water. The climatologist is interested in the relative evaporation in different regions. The botanist, the forester, the plant physiologist and the agriculturist each have still a different interest in the evaporation—as for example, from soil of whatever character and condition of cultivation, from transpiration, from vegetation in whatever form and stage of growth, etc. Thus far no standardized method has been evolved that permits determining exactly the data desired for any one of these requirements, much less a scheme for collecting data generally applicable to all. The plan contemplated by the Weather Bureau aims only to secure data of evaporation under conditions that are so nearly similar in different localities as to yield results having a comparable value. As the work progresses and as the data are examined and discussed in conjunction with observations with which the measurements of evaporation will be correlated, it is hoped to evolve relations that will be of value to the engineer, the forester, the plant physiologist, the botanist and the like.

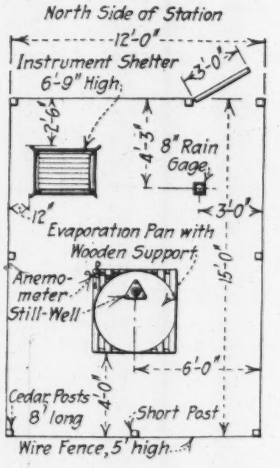


FIG. 1. PLAN OF STANDARD EVAPORATION STATION

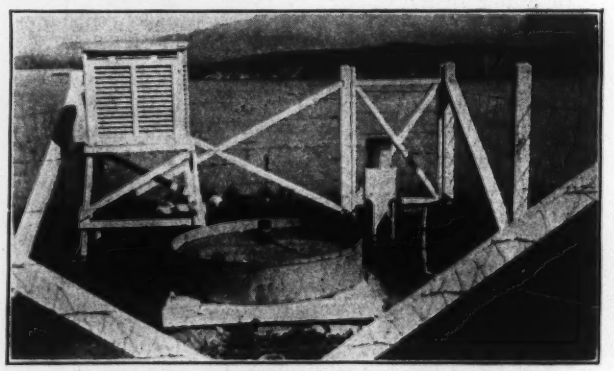


FIG. 2. EVAPORATION STATION AT ROOSEVELT DAM

Ordinarily the observations will be limited to measurements of evaporation, supplemented by such meteorological observations as will serve to identify the climate

*United States Weather Bureau, Washington, D. C.

[1]Described and illustrated by one of us in "Instructions for the Installation and Operation of Class A Evaporation Stations," Circular L, Instrument Division (Benjamin C. Kadel, Chief), United States Weather Bureau, Washington, D. C.

of the region—that is, the climatic records will be made to harmonize with those made by the coöperative observers of the Weather Bureau. An anemometer will be added to the outfit, because of the importance of the wind as a factor in evaporation measurements.

Fig. 1 is a standard plan of a complete observation station, Class A, and Fig. 2 is a view of a station.[2] The standard pan is a cylindrical galvanized-iron vessel 4 ft. in diameter and 10 in. deep, mounted on a skeleton framework just above the surface of the ground. This form of evaporating vessel is undoubtedly subject to some criticism because it is exposed to radiation from sun and sky, but it appears to be the most suitable form when due consideration is given to the ease with which it may be cleaned and cared for, its security against unwarranted change in water level due to deformation and its safety from insidious leaks. The pan is as large as can be safely handled by one man.

A decided improvement in the methods of measuring water heights consists of a heavy-based still well whose

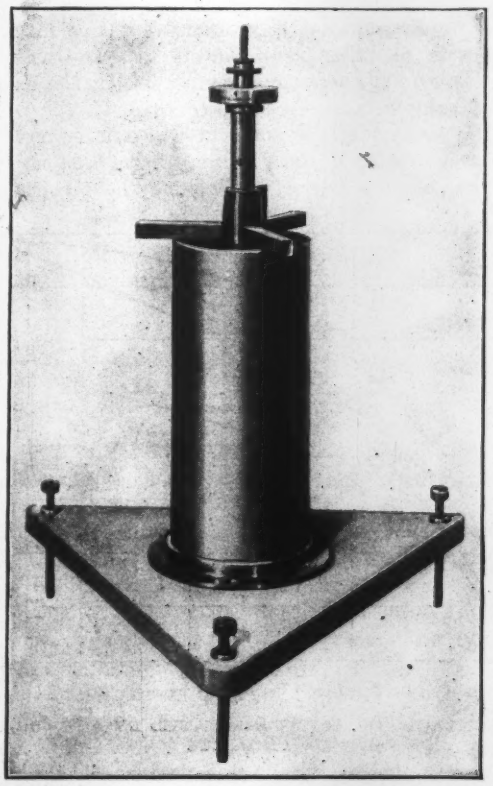


FIG. 3. EVAPORATING-PAN STILL WELL WITH HOOK GAGE

The hook gage is slightly below the top of the still well and is adjustable by a micrometer screw

top forms a fixed point from which measurements are made with a hook gage adjustable by means of a micrometer screw (Fig. 3). Successive readings made by different observers agree within two or three thousandths of an inch, even though observers have never used the gage before. This device, combining simplicity with precision, makes possible satisfactory measurements by untrained men.

[2]For an unidentified view supplied by the authors there has been substituted a view from a photograph of the station at the Roosevelt dam, Arizona, taken recently by a member of the editorial staff of "Engineering News."—Editor.

This simple fundamental, known as Class A station, will be established side by side with floating pans or with the 8-ft. pan buried in the ground, which is the standard of the Bureau of Plant Industry, where opportunities occur. Some 25 stations are expected to begin records this year. These are located throughout the country.

Fair Competition Required in Kansas Bridge Contracts

In the letting of county bridge contracts in Kansas it has been common for the County Commissioners to adopt general plans giving merely the span and width with general specifications for the quality of materials, etc., and invite the bidders themselves to submit detail plans and specifications for the bridge which they propose to furnish.

Under this method of letting contracts the vendor of patent bridges who could make broad claims for the superiority of his designs has had every advantage. Few counties employ engineers competent to give advice on the comparative merits of different designs or even on the safety of a design. As a result of these conditions the chances of real competition in the lettings have been small.

The Supreme Court of Kansas by a recent decision has swept aside this corrupt system of bridge lettings and has ruled that the statute requiring all contracts to be awarded on a public letting to the lowest responsible bidder makes it necessary that the plans and specifications adopted and placed on file for the inspection of bidders must, so far as the nature and character of the proposed work will admit, be sufficiently definite and explicit to enable bidders to prepare their bids intelligently on a common basis. Further, the plans and specifications must be such that the amounts of material required to be used may be determined with reasonable accuracy by all the bidders, and this could not be the case if bids were accepted on designs prepared by one of the bidders.

In the case before the court which gave rise to this decision, the plans and specifications adopted by the Commission and on which it received bids did not give the thickness of the arch ring, the thickness of the piers, the dimensions of the abutments, or the thickness of the wing walls. The opposite side of the case was conducted by a company, which had submitted a bid for a patent bridge, and it contended that these dimensions were dependent on the method of reinforcing which the successful bidder might submit with his plan.

In passing upon this question the Court cited a large number of decisions and said:

> **Authorities are practically unanimous in holding that there can be no active, intelligent competition among bidders unless plans and specifications are open to inspection which are sufficiently explicit as to afford to all bidders an equal opportunity to bid upon the same project or proposition.**

The Court further ruled that after a contract for a bridge has been awarded, it cannot lawfully be changed by additional specifications furnished by the contractor in which substantial changes are made from the requirements on which the other bidders made their estimates.

The case in question was regarded as a test case and was brought by the Topeka Bridge and Iron Co. against the Board of County Commissioners of Labette County, Kan. The case for the county was conducted by the attorney general of the state and the principal witness was Prof. W. S. Gearhart, State Engineer.

Slow Rate of Utilization of Irrigation Works

By R. P. Teele*

SYNOPSIS—An analysis of unpublished Census material to show the average rate at which irrigation works have been put in use. The slowness with which the land in the projects comes into use indicates that something must be done to hasten settlement and to aid settlers in getting their lands under cultivation.

It is well known that irrigation enterprises of any considerable size have been financial failures, almost without exception. A comparison of the values of unimproved arid land and the values of highly improved irrigated land shows a very wide margin which has appeared to be within the reach of builders of irrigation works, but which has so far always eluded their grasp.

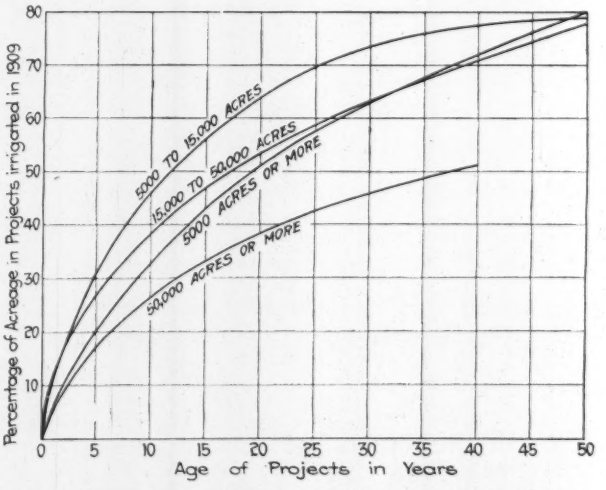
Much thought has been given to the search for the reason for the failure of promoters and builders of irrigation works to realize their dreams. The writer, with many others, has come to the conclusion that the immediate cause is the slowness with which the works built are utilized. This, of course, is not the fundamental cause, for there must be some underlying reason for the almost universal slowness in use. In this article the writer makes no attempt to get at the fundamental cause but he attempts to determine the average rate at which such projects have been utilized, in the hope that a knowledge of what may be expected in this regard may prove helpful in devising plans to overcome this weakness in our plans for reclaiming waste lands.

With the exception of the Government projects and perhaps a few others built within the last few years, no reliable records of the rate of extension of the acreage irrigated under individual projects are available. Very few of the projects for which records may be had have passed the construction stage and consequently their records are of little value for the purpose in hand. The unpublished records of the Thirteenth Census do, however, supply some information on this subject, and this article is based on an analysis of this Census material.

What the Census Figures Show

The schedules used in the special census of irrigation called, among other things, for the year in which each project was begun, the acreage included in each project, and the acreage which was irrigated in 1909 on each project. It is apparent that this will show for each project its age and the extent to which the acreage under it had been placed under irrigation in 1909. Not all of the schedules were completely filled out, but in the preparation of this paper all projects for which the necessary information was given have been tabulated in such a way as to show age and percentage of acreage in projects which were irrigated in 1909. All projects have then been segregated into age groups and the percentage of the total acreage in each age group which was irrigated in 1909 has been computed. The tabulations included all projects which contained 5,000 acres or more, and these have been divided into size groups. The results have been plotted with the age in years on the horizontal scale and the percentage irrigated on the vertical scale, and curves have been drawn to represent the rate at which the land in irrigation projects has been, and presumably will be, irrigated.

It is realized that the data upon which the curves are based are not above criticism, as in many cases the acreage included in projects undoubtedly represents the desires of promoters rather than careful surveys, and the acreage reported as irrigated represents a guess. However, it is believed that the acreages given as in projects are as accurate as the average estimates made by engineers in planning new projects today. One other more or less serious objection presents itself. The method followed assumes that each project had, in the census year, just arrived at the stage in which it is reported, while, as a matter of fact, it may have been as fully utilized for some years, or, in fact, may have gone back.



PERCENTAGES OF ACRES IRRIGATED BY AGE GROUPS OF PROJECTS

For the reasons given no attempt at mathematical refinement in locating the curves has been made. The points used for this purpose are based on percentages computed from the total acreages in projects and the total acreages irrigated for five-year groups running 1 to 5, 6 to 10, etc. In general the points representing the group 11 to 15 years old (1894 to 1898) have not been given as great weight in locating the curves as the other points, since few projects were begun during these years and consequently these points represent much smaller acreages than the other points. In fact, in some size groups there are no projects in this age group.

The total acreage in the projects begun each year since 1858 and the acreage irrigated in 1909 under these projects are given in Table 1 on the next page.

*Chevy Chase, Md.; formerly Special Agent in Charge of Irrigation, Bureau of the Census, Washington, D. C.

TABLE 1. UTILIZATION OF IRRIGATION WORKS

Year Begun	Age in Years	Acreage in Projects	Acreage Irrigated in 1909
1908	1	572,410	61,192
1907	2	910,895	100,271
1906	3	575,518	53,190
1905	4	527,754	80,450
1904	5	974,400	119,740
1903	6	960,236	206,380
1902	7	197,000	78,250
1901	8	257,479	134,785
1900	9	241,087	74,058
1899	10	64,607	23,400
1898	11	29,525	16,550
1897	12	13 169	7,200
1896	13	88,200	53,660
1895	14	151,963	62,710
1894	15	209,356	102,942
1893	16	77,700	54,915
1892	17	183,607	98,350
1891	18	449,799	152,751
1890	19	496,513	206 933
1889	20	310,816	178,157
1888	21	368,362	176,458
1887	22	279,920	125,004
1886	23	242,718	184,513
1885	24	306,492	148,999
1884	25	152,899	100,515
1883	26	180 243	126,912
1882	27	139,180	93 851
1881	28	414,078	188,422
1880	29	142,229	89,521
1879	30	102,505	42,770
1878	31	131,880	88,746
1877	32	245,290	112,239
1876	33	25,350	18,500
1875	34	199,280	100,621
1874	35	86,200	63,000
1873	36	186,815	128,130
1872	37	157,948	59,953
1871	38	221,142	102,632
1870	39	357,549	301,528
1869	40	100 296	70,986
1868	41	6,400	2 560
1867	42	8,000	3,500
1866	43	25,730	19,520
1865	44	34,223	28,552
1864	45	68,270	63,270
1863	46	28,000	14,997
1862	47	21,055	19,055
1861	48	19 570	12,530
1860	49	67,854	59,860
1859	50	10,000	10,000
Total	..	21,621,512	4,423,028

It will be noted that these projects include a total acreage of 21,621,512 acres, or about two-thirds of the total acreage included in all projects reported by the Census. The acreage irrigated in 1909 under these projects, however, was but 4,423,028 acres, or about one-third of the total acreage irrigated in that year, according to the Census. This wide difference in proportion is explained by the fact that the largest acreage is represented by projects from one to six years old, which show low percentages irrigated.

Incidentally, Table 1 shows much of the history of irrigation development in the United States. While the figures in the table represent only a part of the projects it is probable that they indicate correctly the periods of activity in large-scale irrigation development and the periods of depression. It will be noted that the first big year was 1870, the year when several of the enterprises in northern Colorado were begun. This activity continued for a few years, to be followed by a period of depression after the panic of 1873. Development soon revived, however, and there was activity until the panic of 1893, when there was a serious setback which showed its greatest effect in the years 1896 to 1899, the year 1897 showing a smaller acreage in projects undertaken than any year since 1868, almost thirty years before. A revival began in 1900, which was greatly accelerated by the passage of the Reclamation Act in 1902, the years 1903 and 1904 showing the largest areas in the entire period. This activity continued up to the census year (1909) but began slowing down at just about that time. As is well known, there has been an almost complete cessation in the beginning of new projects since that time.

The accompanying curves show the rate of utilization of all projects embracing (1) 5,000 acres or more, (2) from 5,000 to 15,000 acres, (3) 15,000 to 50,000 acres and (4) 50,000 acres or more.

As is to be expected, the smaller projects show the most rapid rate of utilization and the larger projects the least rapid rate with the intermediate group and the average for all projects lying about midway between the two. Table 2 shows in percentages, on the basis of the charts, the extent to which the projects of the different size groups may be expected to be utilized at the five-year intervals after their beginning.

TABLE 2. PERCENTAGE OF ACREAGE IN PROJECTS IRRIGATED AT FIVE-YEAR INTERVALS FROM DATE OF BEGINNING

Years after beginning	5	10	15	20	25	30	35	40	45	50
	Percentage Irrigated									
All projects	20	33	43	51	58	63	68	72	76	80
5,000 to 15,000 acres	31	46	56	64	70	74	76	78	79	80
15,000 to 50,000 acres	29	40	47	54	59	63	67	71	75	78
50,000 acres or more	18	27	34	39	43	46	49	51	..	..

In the light of these figures it is not at all difficult to see why irrigation projects have been financial failures. It is customary to make rather liberal estimates of the acreage which can be irrigated by a proposed project and rather close estimates of the cost, then spread the cost over the entire acreage, and base the selling price of rights per acre on the ratio between this underestimated cost and the overestimated acreage, with, of course, a suitable margin for profit. The curves indicate that even 50 years after the beginning of the average project only 80% of the land is irrigated and contributing to repaying the cost, while the percentage for the larger projects is much lower. At the end of 20 years only 50% of the land in the average project and 40% for the large project is producing. In the mean time interest and operation and maintenance charges for the balance of the land must be carried by the constructing agency or by the land which is producing. In the one case it will break the company, and in the other it will break the farmers—and that will break the company.

Obviously, it will be necessary to increase greatly the selling price of water rights or take steps to accelerate the rate of utilization; and if settlement is slow with a low price for water it will presumably be slower with a higher price. It seems apparent, therefore, that the road to successful irrigation development lies along the line of giving greater attention to the settlement of the lands included in irrigation projects and to aiding settlers in bringing their lands into cultivation. It is not the intention to discuss these subjects in this article.

Water Consumption Has Been Reduced at Northampton, Mass., by the combined use of the pitometer and service inspections, according to the 1915 report of the Water Commissioners (Luther C. Wright, Supt.). The use of the pitometer in November, 1914, showed an average daily consumption of 2,851,000 gal. and a minimum night rate of 1,889,000 gal., or a night rate 66% of the day rate. A house-to-house inspection to detect leaking fixtures was made. In April, 1915, after only some 50% of the services had been inspected, the daily consumption was 2,663,000 gal., the minimum night rate 1,560,000 gal., thus giving a night rate which was 58% of the day rate and an actual reduction in night consumption of 329,000 gal.

Kansas Road Maintenance by Contract System

By E. V. Moone*

Finding it difficult to get satisfactory dirt-road maintenance done by farmers living along the roads, Sedgwick County, Kansas, has been trying out a new plan of road maintenance on the four main roads out of Wichita. Each one of these radiating roads has been placed under the charge of a competent man, who, under contract, devotes his entire time to maintenance and patrol, hires additional help when needed, and who holds his job only so long as results obtained justify the money expended—in the judgment of the county commissioners and the county engineer.

These men are under contract to keep the road in repair in every way, dragging, grading, resurfacing, filling around bridges and culverts, cutting weeds and trimming hedges, etc. While the amount of money expended is larger than the usual Kansas county dragging allowance, the results have been enough better to seem to warrant the additional expenditure. The better maintenance has concentrated so much travel on these main roads that the additional expense seems fully warranted. Travel on these roads frequently averages a motor car a minute during the day and early evening.

On the road east of Wichita, a farmer who has several boys and plenty of teams, as well as an automobile, is the contractor. This road leads to the newly opened Augusta oil fields and recently has been much traveled. Although of heavy soil, with no surfacing, this road has been in excellent condition almost every day, despite heavy rains and resulting floods.

The road north of Wichita has 13 mi. covered with Little Arkansas River gravel, and the cost of maintenance here has been kept down materially. The contractor on this section finds time to use some of his four teams on farm work and occasional hauling. South of the city is 9 mi. of sand-clay road and about 4 mi. with gravel surfacing. Two men and four teams work overtime on this road as it requires much hauling of clay for patching, as well as a great deal of "dressing up" with a grader.

West of Wichita one man is trying to maintain 24 mi. of road with a Nielson tractor. Although he can pull two drags and frequently makes the 48-mi. round trip in a day, he has not had entire success with the work. The difficulty of getting teams to do the necessary work that cannot be done with a tractor has been one of the main objections, and the county authorities are seriously considering dividing the road into two sections under two patrolmen.

The contractors are furnished with steel road drags, small graders, slip scrapers, shovels, axes and other necessary small tools. They contract to work under the direction of the county engineer, and to devote all the time necessary for proper road patrol and maintenance, and to keep themselves informed by actual inspection of the road as to its condition at all times. They are paid a stipulated flat rate per hour, which is a little less than average wages for men and teams and for single hands in this locality. Patrolmen sign receipts for county machinery in their charge and report all work done to the county engineer on post cards within 24 hr. of the time it is finished.

*County Engineer, Sedgwick County, Wichita, Kan.

This system of maintenance gives the county authorities a chance to hold some one man responsible for the road condition on the section under his charge and eliminates the endless struggle to get the farmers to neglect their own work for the sake of the road, when both farm and road demand attention simultaneously. Results are generally approved by the people of the county, and no objections have been raised on account of the additional expense.

Pumping 7,500,000 Gallons Daily by Air Lift

The entire water supply (7,500,000 gal. per day) of El Paso, Tex., a city of 100,000 population, is raised from 32 deep wells by an air-lift system, making this probably the largest and most notable installation of its kind in the country. The water-works were taken over by the city from the International Water Co. in 1910 and since then improvements have been made under the supervision of Frank H. Todd, Consulting Engineer for the city.

The wells are 8 to 14 in. in diameter and 500 to 600 ft. deep. They are sunk in a high mesa north of the city, near Fort Bliss. The mesa is a part of the noted Tularosa Valley (the underground water-supply of which is the subject of a recent United States Geological Survey paper). These wells pass through irregular beds of clean but very fine sand with interbedded layers of clay. The latest wells have been driven by the clay-puddling method, in which no drill casing is used. The well casing is standard wrought-iron or steel pipe, with vertical rectangular slots 3/8x10 in., cut out of the sections of pipe that pass through the water-bearing strata. These cuts are made by the water-works employees on the job. The method of placing the air line, shown in the accompanying sketch, is new and was devised by Mr. Todd. The air line is 2 in. and the water line 6 in. in the new 12-in. wells. The air inlets are about 214 ft. below the ground surface. The level of the water in the wells has gone down from about 177 ft. in 1905, when the first wells were sunk, to about 200 ft. at the present time. Each (12-in.) well yields about 450 gal. per min.

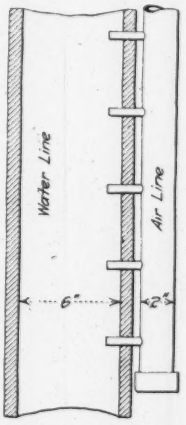


DIAGRAM OF AIR-LIFT SYSTEM

The air-compressor plant consists of two old steam-driven compressors, one a 750-cu.ft. Ingersoll-Rand and a 2,600-cu.ft. Ingersoll-Sergeant. The new equipment added since the city took over the works includes a 3,600-cu.ft. Allis-Chalmers steam-driven unit and two 2,000-cu.ft. Diesel-engine Nordberg compressor units. The Diesel engines are each 500-hp., Fulton Toisi engines, built by the Fulton Iron Works, St. Louis, Mo. Each is direct-connected to a 2,000,000-gal. triplex pump on one side and a 2,000-cu.ft. air compressor on the other. The compressors are driven at 155 r.p.m.

Water lifted by the air is delivered by concrete conduits to a centrally located basin where it flows to the pump intakes over a measuring weir. The water supply is then delivered into the city mains by high-pressure pumps.

Bituminous Materials Tests for Sheet Asphalt

By Francis P. Smith*

SYNOPSIS—What tests for bituminous materials should be included in sheet-asphalt specifications? A discussion of points raised in an alleged attack on the specifications adopted in 1915 by the American Society of Municipal Improvements.

A lecture was delivered before the graduate students in highway engineering at Columbia University, on Jan. 13, 1916, by Charles N. Forrest, Chief Chemist of the Barber Asphalt Paving Co. This lecture has been printed in full or in abstract in various places since that date. It describes and compares most of the tests that are applied at the present day for the examination and identification of bituminous materials.

In so far as the paper relates to methods and results, it contains much of value. It also contains what the writer considers an unwarranted attack upon the specifications adopted by the American Society of Municipal Improvements in 1915. As the writer has been chairman of the Committee on Sheet-Asphalt Specifications of that society for a number of years, he feels called upon to reply thereto. The following are quotations from Mr. Forrest's paper:

> The variety of bituminous materials available for road-building purposes is now very much greater than it was a decade ago. The manner of their application has also enlarged and the suggestion of new methods for testing them has added a number of additional features to the earlier schemes, but even so, there are few tests employed in the general examination of bitumens which can be said to directly indicate quality for any specific purpose. Some of them are very properly referred to as indirect indications of quality, but after all, as the subject now stands, the only real test of quality still remains a service test over a period of years.
>
> Following the introduction of the many new products or so-called paving bitumens, during the past ten years, each one of which has generally appeared with many claims of superiority, there has been a marked tendency to contract the general scheme of testing which was in vogue prior to their introduction, in order that those characteristics which such schemes indicated were essential, but did not happen to fit some specific new product, could be ruled out.
>
> The deliberations of associations of learned societies which have had such matters as the drawing of specifications and schemes for testing materials under consideration, and are necessarily obliged to assume a neutral position, have resulted in, first, the elimination of all features which are not common to all substances of a certain class which have not been an absolute failure in service and, second, the extension of the limits of specifications to include everything which has any sort of a favorable indorsement.
>
> The recommendations of such bodies are always a compromise and generally reflect average, rather than the most advanced thought or practice.
>
> As an illustration of the considerable abbreviation of present-day specifications, the recommendations of the American Society of Municipal Improvements for suitable asphalt for sheet-asphalt pavements may be quoted. It is as follows:

SPECIFICATIONS OF AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS, 1915

Refined Asphalt	Asphaltic Cement	Flux
Prepared from natural mineral bitumen	Refined asphalt and flux	Residue of petroleum
Penetration, 77° F. Solubility CCl_4	Penetration, 77° F. Flash Volatilization Hardness after volatilization Ductility	Penetration, 77° F. Specific gravity Volatilization Flash Solubility CCl_4

> This is a typical example of a compromise specification and one which is popularly described as open, because a great variety of substances fall within the scope of its limitations, both in regard to common characteristics and the wide latitude of the few of them which have been mentioned.
>
> A more elaborate specification or scheme of examination is illustrated in the form of report of the Office of Public Roads for bituminous materials for different purposes.

This quotation is preceded by a scheme of tests which were stated to have been adopted by Mr. Richardson for the *identification* of asphalts. In order to avoid reproducing all three of these schemes, herewith is given an enumeration of all the tests in Mr. Richardson's schedule and also in the two schemes of the Office of Public Roads:

> Physical Properties—(1) Specific gravity, (a) original substance dry, (b) pure bitumen; (2) color of powder or streak; (3) luster; (4) structure; (5) fracture; (6) hardness original substance; (7) odor; (8) softening point; (9) flows; (10) penetration at 78° F.; (11) general character; (12) melting point; (13) flash point; (14) burning point; (15) viscosity; (16) float test; (17) ductility.
>
> Chemical Characteristics—(18) Loss on heating, (a) original substance, (b) dry substance; (19) character of residue; (20) penetration of residue; (21) bitumen soluble in carbon disulphide; (22) organic matter insoluble; (23) mineral matter; (24) bitumen soluble in 62° Bé. naphtha; (25) bitumen soluble in 88° Bé. naphtha; (26) per cent. of bitumen removed by H_2SO_4; (27) bitumen soluble in carbon tetrachloride; (28) fixed carbon; (29) sulphur; (30) distillation test; (31) ultimate composition.
>
> Additional tests which have been used to some extent and which, though mentioned in the article, are not included in the schemes referred to are as follows: (32) Paraffine; (33) brittleness.

It is evident that these schemes of analysis are meant to be broad enough to embrace the examination of all classes of bituminous materials, both in their raw state and in their manufactured state. It therefore follows that many of these tests do not apply to the examination of refined asphalts or asphalt cements for use in sheet-asphalt pavements and should not be included in sheet-asphalt specifications. Among the tests enumerated there would appear to be no question that the following are for identification purposes only:

> (1-b) Specific gravity, pure bitumen; (2) color of powder or streak; (3) luster; (4) structure; (5) fracture; (6) hardness original substance; (7) odor; (11) general character; (18-a) loss on heating, original substance (as differentiated from dry substance); (22) organic matter insoluble; (23) mineral matter; (26) per cent. of naphtha-soluble bitumen removed by H_2SO_4.

Certain other of these tests have never been seriously advocated for use in sheet-asphalt specifications, as follows:

> (8) Softening point; (9) flows; (14) burning point; (15) viscosity; (30) distillation test; (31) ultimate composition.

Softening point and flow point, if of any value from the sheet-asphalt paving standpoint, are covered by melting point, which latter test is hereinafter discussed. Burning point is obviously valueless. Viscosity is of certain value in connection with road oils, but is dependent upon so many conditions as to be chiefly of use as an identification test.

As far as the writer's view of the distillation test is concerned, it entirely coincides with the following sentences contained in Mr. Forrest's paper:

> As a research matter, distillation in the laboratory of all kinds of bituminous materials has been a source of considerable information, but as a routine test is impractical in the examination of road-building materials except under certain tar and cut-back oil products.
>
> The same is true with regard to ultimate composition.
>
> The ultimate analysis of bitumens and other road-building materials has never been of interest to practical road builders.

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Included among the remaining tests are certain others that may be regarded as controversial. These are as follows:

(1-a) Specific gravity, original substance dry; (12) melting point; (16) float test; (24) bitumen soluble in 62° Bé. naphtha; (25) bitumen soluble in 88° Bé. naphtha; (28) fixed carbon; (29) sulphur; (32) paraffin; (33) brittleness.

With regard to the specific-gravity test, Mr. Forrest appears to agree with Professor Blanchard that it is of value mainly as a means of identification.

While in a few instances melting-point and viscosity requirements have been incorporated in sheet-asphalt specifications, the use of these tests and the advocacy of them have been so limited that the writer believes he is justified in dismissing them from further consideration.

The float test is in a sense a measure of consistency, giving information which occasionally is useful in connection with viscosity, melting-point and penetration tests. It can hardly be deemed essential in sheet-asphalt specifications, however, and its inclusion in them has never been seriously advocated.

While certain authorities have urged the use of the naphtha-solubility test as indicating the amount of body-forming hydrocarbons, its inclusion in specifications has been largely for purposes of identification. The test itself appears to have been originated in the early days of the industry, chiefly as a means for distinguishing between so-called Trinidad Lake and Land asphalts. The limits of solubility in naphtha between which asphalts could be considered as suitable for paving purposes and outside of which they should be condemned for paving purposes have never been established.

The writer believes that the test for fixed carbon is valueless as indicating the suitability of an asphalt for paving purposes.[1]

Sulphur has been recognized as playing a very important part in the formation of bitumens in nature.[2] In the early days of the industry no specifications made any reference whatever to sulphur contents. Trinidad and Bermudez asphalts contain approximately 5.5% and 4% of sulphur respectively. About the time that California asphalt was put upon the market and became a serious factor in the paving industry certain specifications were promulgated containing clauses to the effect that the asphalt must contain at least 2% of sulphur not artificially incorporated. As California asphalt contained only traces of sulphur, this was evidently an attempt to bar out California asphalt.

The history of the paraffin test is somewhat similar. Many of the Mexican asphalts are higher in paraffin than those obtained from California, Trinidad or Venezuela; and regardless of the fact that asphalt cements for many years had contained large proportions of paraffin flux, an attempt was made to bar out Mexican asphalts by inserting a clause in the specifications to the effect that the asphalt should contain not more than 1% of paraffin. This clause, if adopted, would permit the use of California, Trinidad and Bermudez asphalts, but would bar out many Mexican asphalts. The subject of the paraffin contents of an asphalt was treated exhaustively by A. W. Dow and the writer in an article that appeared in *Engineering News* of June 8, 1911, pages 680 to 683.

As far as the brittleness test is concerned, Mr. Forrest states in the article under discussion, as follows:

A test for brittleness is not usually mentioned in connection with bituminous road-building materials, but is in use by some investigators for special purposes.

It would appear from this that Mr. Forrest did not have any intention of urging the inclusion of this test in sheet-asphalt specifications.

Based on the foregoing, it is the writer's opinion that the tests just discussed as identification tests, as tests not suitable for the examination of asphalt cements for paving purposes and as controversial should not be included in sheet-asphalt specifications. This leaves the following tests available for the examination of paving asphalts, *all of which are included in the American Society of Municipal Improvements specifications:*

(10) Penetration; (13) flash point; (17) ductility; (18) loss on heating; (19) character of residue; (20) penetration of residue; (21) bitumen soluble in carbon disulphide; (27) bitumen soluble in carbon tetrachloride.

The writer believes that he is entirely justified in stating that the only inference possible for the uninformed hearer or reader to draw from Mr. Forrest's paper is that the American Society of Municipal Improvements specifications have had so many important tests omitted from them that they are valueless and that Mr. Forrest has attempted to support this view by marshaling an imposing array of tests, regardless of whether or not all of them are really applicable to refined asphalts or asphalt cements for use in sheet-asphalt pavements. In his attempt to belittle the American Society of Municipal Improvements specifications he has misquoted them and omitted tests that they contain, as is shown by the following quotation.

Under the head of "Refined Asphalt" the A. S. M. I. specifications contain:

a. Clauses governing the methods of refining and providing for supervision of them and inspection at the refinery.

b. Every refined asphalt admitted under these specifications, if required by the engineer, shall be equal in quality to the recognized standard for its particular kind or type of asphalt.

c. Asphalts obtained by the refining of natural liquid bitumens shall not be reduced in the refining process to a penetration at 77° F. of less than 30.

d. All shipments of refined asphalt of any one kind shall have the batch number plainly marked on each package or container and shall be uniform in consistency and composition and shall not vary from maximum to minimum more than 15 points in penetration at 77° F.

e. A clause for solubility in carbon disulphide and carbon tetrachloride (and not only carbon tetrachloride, as stated).

The requirements for asphalt cement are correctly stated, although no mention is made of the clauses safeguarding its preparation and handling. The clause limiting the materials from which fluxes may be made is omitted, as well as the following clause:

They shall be of such character that they will combine with the asphalt to be used to form an acceptable and approved asphalt cement complying with the requirements of these specifications.

As further justification for the writer's criticism of Mr. Forrest's paper the following is a complete list of the tests contained in sheet-asphalt specifications drawn up, distributed and advocated by the company of which Mr. Forrest is chief chemist. For convenience of comparison a parallel column shows the list included in the A. S. M. I. sheet-asphalt specifications:

[1] Those interested in an exhaustive discussion of this test should refer to "Municipal Engineering," September, 1913, and "Canadian Engineering," Nov. 13 and 20, Dec. 4 and 18, 1913; Jan. 1 and 15 and Feb. 5, 1914.

[2] Those who are interested in this matter may refer to Mr. Richardson's article on the subject appearing in the "Journal" of the Society of Chemical Industry, January, 1898.

Barber Asphalt Paving Co.	American Society of Municipal Improvements
Flux:	Flux:
Water	.....
Specific gravity	Specific gravity
Penetration	Penetration
Solubility in carbon disulphide	Solubility in CCl_4
Flash test	Flash test
.....	Loss on heating
.....	Character of residue
Refined Asphalt:	Refined Asphalt:
Water	.....
Penetration	Penetration
Solubility in CS_2	Solubility in carbon disulphide
Fixed carbon	.....
Paraffin scale	.....
Sulphur	.....
.....	Solubility in carbon tetrachloride
.....	Uniformity of penetration and composition of different shipments
Asphalt Cement:	Asphalt Cement:
Specific gravity	.....
Penetration	Penetration
Loss on heating (penetration and ductility of residue)	Loss on heating (penetration of residue)
Solubility in CCl_4	Solubility in carbon tetrachloride
Organic insoluble (nonbituminous material)	.....
Flash test	Flash test
Float test	.....
Paraffin scale	.....
Ductility	Ductility
Fixed carbon	.....
Uniformity of penetration and ductility of different shipments	Uniformity of penetration and composition of different shipments
.....	Solubility in carbon disulphide

From the foregoing we see that the A. S. M. I. specifications are more exacting on the subject of flux than are the B. A. P. Co. specifications. While the water test is not specifically mentioned in the A. S. M. I. specifications, as it has but little bearing on the quality of the material, it is implied owing to the fact that a flux containing any considerable proportion of water could not be used in the kettles without causing foaming.

Taking up the subject of refined asphalt, the same remarks apply to the water test mentioned thereunder. The A. S. M. I. specifications include a test for solubility in carbon tetrachloride and a provision for insuring the uniformity in penetration and composition of different shipments. These tests are not included in the B. A. P. Co. specifications. The tests for fixed carbon, paraffin scale and sulphur included in the B. A. P. Co. specifications and not in the A. S. M. I. specifications are either controversial or purely identification tests and have been dealt with at length in a previous portion of this communication.

As far as the tests required for asphalt cement are concerned, we find that the B. A. P. Co. specifications contain the following tests which are not present in the A. S. M. I. specifications: Specific gravity, nonbituminous matter, float test, paraffin scale, fixed carbon.

These again are chiefly identification tests. As previously explained, they are in no sense indicative of the paving value of the asphalt. As a matter of fact, the "nonbituminous-matter" test is so drawn as to exclude anything but Trinidad and Bermudez asphalts. The paraffin-scale test is so drawn as to exclude a large proportion of the Mexican asphalts. The fixed-carbon test is so drawn as to exclude practically all of the California, Texas and Mexican asphalts. There can be no question, therefore, that these tests are inserted purely for purposes of identification and limitation. This is also true of the fixed-carbon, paraffin-scale and sulphur tests under refined asphalt. The value of the float test is very problematical.

In considering the foregoing it must be borne in mind that the Barber Asphalt Paving Co. was under no obligations to assume a neutral position, and the fact that it deliberately included tests which barred out asphalts other than Trinidad Lake and Bermudez asphalts clearly showed that it was not its intention to assume a neutral position. It therefore was free from any of the limitations which Mr. Forrest insists necessarily attend the deliberations of learned societies. In spite of this fact the only tests present in its specifications which are not included in the A. S. M. I. specifications are such tests as would identify certain asphalts and admit them to the exclusion of others. Further comment along these lines would appear to be unnecessary.

After discussing the various tests in detail, Mr. Forrest further states:

> Prof. A. H. Blanchard in his very excellent book, "Elements of Highway Engineering," has quoted Mr. Prevost Hubbard upon the subject, "Interpretation of Results of Tests," and the following remarks from that source upon the significance of a number of tests which are now under discussion are of interest:
>
> Specific gravity is of value mainly as a means of identification. . . .
>
> Flash point is of little value, other than a quick means of differentiating between heavy crude oils and cut-back products and fluid residuums . . . .
>
> Melting point of solid bitumen is mainly of value as a means of identification, and for control work on the part of manufacturers.
>
> Maximum and minimum limits covering consistency are essential. (Consistency as here referred to embraces penetration or viscosity.)
>
> The distillation test as applied to tars is a very valuable one both for the purpose of ascertaining their road-building properties and method of preparation if they are refined products.
>
> Solubility Carbon Disulphide—The solubility of bitumen itself is entirely independent of its character and consistency.
>
> Solubility 88° Bé. Naphtha—As applied to oils and oil products, this determination is of value as indicating the amount of body-forming hydrocarbons which give mechanical stability to the material.
>
> Evaporation test is a purely arbitrary one, but when applied to road oils will often prove of considerable value.
>
> Fixed carbon shows much the same thing as naphtha-insoluble bitumen . . . .
>
> Supplementing the above estimate of the value of certain tests by Mr. Hubbard, a quotation from Messrs. Dow and Smith[3] upon the subject of paraffin is given by Professor Blanchard as follows:
>
> Paraffin—We are decidedly of the opinion that the scale paraffin test per se should not be regarded as a measure of value of bituminous compounds for road-making or paving purposes.
>
> Upon inspection of the specifications of the American Society of Municipal Improvements for asphalt cement for sheet-asphalt pavements it will be observed that but few of the tests which have just been discussed are mentioned. Those which are mentioned, or which affect the asphalt cement through being mentioned under the requirements for refined asphalt or flux, which combine to produce the asphalt cement, are penetration, evaporation and ductility. These are characteristics which are common to all bituminous materials and, in the light of the available information upon this general subject, cannot be regarded as furnishing conclusive evidence of suitable quality of any or all materials for such a purpose.

It is entirely true that the characteristics shown by the penetration, evaporation and ductility tests are common to all bituminous materials; but if they were not common to such bituminous materials as are used in the paving industry, it would of course be obvious that they could not be included in the specifications. The fact that they are common to bituminous materials which are not used in the paving industry has no bearing whatever upon the subject, provided the limits prescribed in the specifications bar out such materials as are not suitable for sheet-asphalt paving, and this the writer contends is the case.

Mr. Forrest also states:

> The drift of this subject during the past few years, during which time one after another of the tests which did not happen to suit some new material, as it had theretofore been interpreted, has been thrown into the discard, rather brings us

[3] By Prevost Hubbard—"Proceedings," American Road Builders Association, pp. 213 to 216; see also "Engineering News," June 8, 1911, pp. 680 to 683.

abreast with the fact that a great deal of attention is being paid to the testing of an element which forms but a small part of an article of manufacture—namely, the pavement in place and ready for work—which might perhaps be devoted with the hope, at least, of more positive results upon the finished article as a whole.

The writer contends that a review of the history of the asphalt-paving industry and the specifications which have been adopted from time to time shows that the first portion of Mr. Forrest's statement as just quoted is not in accord with the facts and that, if anything, the reverse is true. Practically the earliest specifications that were adopted called for Trinidad asphalt or its equal. A little later on, a competing asphalt entered the market obtained from the Island of Trinidad, but not from the so-called Pitch Lake. The specifications then underwent an amendment to the effect that the asphalt must be equal in quality to that obtained from the Pitch Lake in Trinidad, and the naphtha-solubility test was advocated as practically the only means of differentiating between these two kinds of Trinidad asphalt. A little later, Bermudez asphalt appeared upon the market, and an active campaign was started to show that it was distinctly inferior to Trinidad Lake asphalt. In fact, Mr. Richardson in a lawsuit in Lincoln, Neb., went so far as to declare on the witness stand under oath that Bermudez asphalt was a most inferior paving asphalt.

When the Bermudez and Trinidad Lake asphalts were gathered under one control, the merged interests waged a joint warfare against the California asphalt and clauses were introduced in specifications to the effect that all asphalts must be made from hard natural asphalts such as were found in the Trinidad and Bermudez deposits. Although the test had never been used in asphalt specifications it was then discovered that the fixed carbon in Trinidad and Bermudez asphalts ran below 15%, while that in the California asphalts ran above 15%; 15% was therefore advocated as the limit of demarcation between good and bad asphalts. The sulphur test previously alluded to was also introduced. Later on, when the Mexican asphalt appeared in the market, the paraffin test was advocated as a very valuable one, as the writer has already described, although it, too, had previously never appeared in any asphalt specifications.

It is quite true that a great deal of attention is being paid to the testing of the asphalt itself and that the asphalt constitutes only approximately 10% of the finished pavement. So far as the A. S. M. I. specifications are concerned, however, an examination of them reveals that a great deal of attention is also paid to the testing and selection of the materials forming the balance of the pavement. It is therefore not correct to draw the conclusion that in the A. S. M. I. specifications only the asphalt is tested and that the tests prescribed are so limited in number and so inconclusive in character that both good and bad asphalts are admitted indiscriminately under them. Broadly speaking, asphalt for paving purposes must meet three essential requirements:

1. It must be of the proper consistency for use.
2. It must be sufficiently cementitious to firmly bind the particles of mineral aggregate together.
3. It must be sufficiently permanent in character to withstand the test of time, traffic and climatic conditions.

The writer contends that the fact that an asphalt is found in a so-called lake has nothing whatever to do with these qualities. Many asphalts are found in deposits of this character which are totally unfitted for paving purposes, and much of the asphalt found in the Bermudez Lake is unfit for paving purposes.

So far as consistency is concerned, that is ordinarily determined with sufficient accuracy by the penetration test. So far as cementitiousness is concerned, the ductility test up to the present has given us more information in this respect than any other test that has been devised and Mr. Forrest is on record as stating that "the ductility test as performed according to the general way of doing it, as we know it, is a measure of the cementitious value."

As an accelerated exposure test, we undoubtedly gain very valuable information by subjecting the asphalt to heating for a stated period of time under stated conditions and noting the hardening due to this heating and the character of the residue left after heating.

In other words, these three tests form the skeleton on which we rely to determine the suitability of an asphalt for paving purposes. Supplementary tests are of value in helping us form an opinion; but where the limits given for these supplementary tests are such as to bar out certain brands of asphalt regardless of their worth as established by service tests and experience throughout the country, their introduction into a specification is indefensible from the public standpoint.

From this brief review of the subject we see that in direct opposition to Mr. Forrest's statement there has been a constant tendency in the asphalt-paving business whenever a new asphalt came up to decry its value and to attempt to devise tests that would show a difference between the new asphalt and those previously used and then advocate this test as a very valuable one and set such permissible limits under it as would bar out the new asphalt.

Quoting again from Mr. Forrest's lecture:

Bituminous pavements which are prepared locally and put down in sheet form—that is, as a unit—do not lend themselves readily to testing as a whole or as a complete mixture; but it is obvious that a series of tests which can be applied to the finished article of manufacture—namely, the whole mixture or the pavement in place—would be much more conclusive and satisfying from the point of view of the engineer who is responsible for the work than the type of tests now in vogue, confined as they are to the raw materials as separate units, and give no direct information as to how congenial such raw materials will be after they are combined.

There is no doubt that a series of tests on the asphalt pavement itself would be valuable, if they could be devised and standardized. Although many efforts have been made in this direction, up to the present they have not met with success, which accounts for their not being incorporated in specifications. The specifications, however, do recognize the variations rendered necessary by differences in the character of the raw materials used and such modifications are placed under the control of the engineer.

The National Road Board of Great Britain has devised a very elaborate machine for testing asphalt pavements, but the results obtained have not yet been reduced to a practical basis and such a machine would be too expensive to install in a city or private testing laboratory.

Finally, it may be said that, recognizing the difficulty of obtaining concordant results in different laboratories and the necessity for employing uniform methods for conducting the various tests, the A. S. M. I. specifications describe each test in detail, state the limit of

accuracy of the test itself and what conclusions may be drawn in each case from the results obtained. They also specify the size of the samples to be examined and the methods to be employed in obtaining them.

[Proof of the foregoing article was submitted to C. N. Forrest, Chief Chemist of The Barber Asphalt Paving Co. Mr. Forrest's comments on Mr. Smith's statements follow.—Editor.]

Sir—I have read Mr. Smith's discussion of my recent address to the graduate students in highway engineering at Columbia University. I fail to find, however, any misstatements on my part in my references to the specifications of the A.S.M.I. It appears that the tests contained in these specifications that I am said to have omitted are really not tests at all, but certain clauses that simply refer to the tests that I did mention. Mr. Smith states in his "e" that "under the head of Refined Asphalt the A.S.M.I. specifications contain a clause for solubility in carbon disulphide and carbon tetrachloride" (and not only carbon tetrachloride, as stated).

Again consulting my copy of the proceedings of the A.S.M.I., 1914, p. 588, par. 4, and succeeding clauses on Refined Asphalt, I fail to find any reference to solubility in carbon disulphide, or any mention of specific tests that I did not quote. I have no criticism to make of this omission in the specifications, but merely make a statement of fact.

It may be of interest to mention in connection with this subject that despite the specifications which Mr. Smith supports so feelingly, a committee on Standard Tests for Bituminous Materials was appointed at the last convention of the A.S.M.I. under the chairmanship of Professor Blanchard "to invite and direct the coöperation of all chemists dealing with bituminous materials for the purpose of further standardizing present methods of testing and for the purpose of encouraging and developing additional methods and appliances for valuating paving bitumens." C. N. FORREST.

Maurer, N. J., June 28, 1916.

Catchbasins and Culverts on County Roads

The extensive system of bond-issue roads now under construction in Vermilion County, Illinois, includes a great number and variety of minor structures in the way of culverts, farm-drain entrances and catchbasins. This work is under the direction of P. C. McArdle, Chief Engineer for the Vermilion County Roads.

The standard catchbasin, Fig. 1, consists of a 24-in. vitrified-tile pipe set on end, with a circular slab of concrete around the bell and a concrete filling at the bottom. A cast-iron grating is set in the bell. The pipe has an outlet of the same diameter as the drain with which it connects. The concrete is 1:3:5.

For farm-drain entrances an 8-ft. length is lined with tile, embedded in concrete and having a concrete cutoff wall, as shown in Fig. 2. The concrete is a 1:3:5 mix, and the quantity ranges from 1½ cu.yd. with 10-in. tile to 3½ yd. with 24-in. tile.

The standard culverts are of reinforced concrete of rectangular box section, with end walls of plain concrete, as shown in Fig. 2. The normal sizes range from 1x1

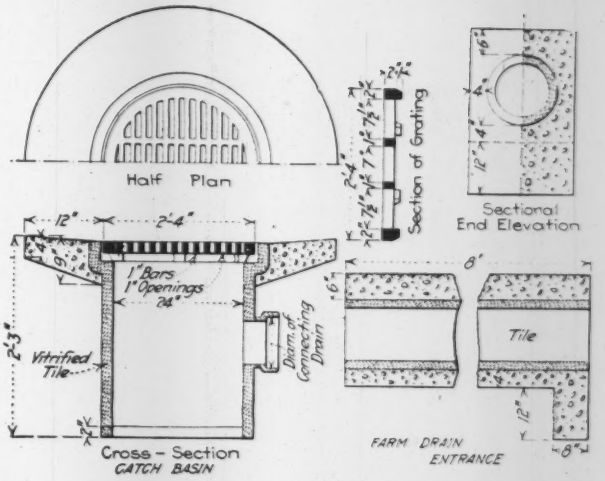

FIG. 1. CATCHBASIN AND FARM-DRAIN ENTRANCE FOR VERMILION COUNTY ROADS, ILLINOIS

ft. to 10 ft. span and 8 ft. high. Special sections are sometimes required, having great width in proportion to height in order to give the necessary area of waterway and to keep within restrictions of headway and grade.

The concrete in top and bottom is 1:2½:4, while that for the side walls and head walls is 1:3:5. All exposed edges are beveled by means of ¾-in. triangular molding set in the forms. The reinforcing bars, which are in the top and bottom only, are of mild or medium steel rolled from billets, no rerolled material or high-carbon

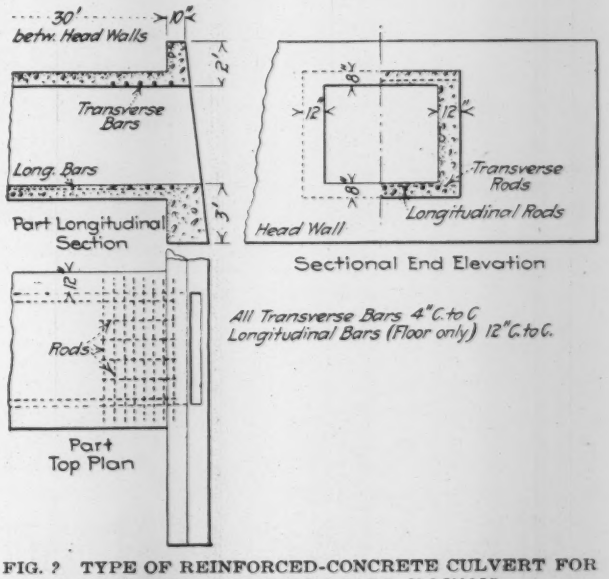

FIG. ? TYPE OF REINFORCED-CONCRETE CULVERT FOR VERMILION COUNTY ROADS, ILLINOIS

steel being permitted. The standard drawings contain lists of quantities, with lengths and sizes of bars, for the various sections of culverts. Tile drains 3 in. in diameter are placed in the abutment walls, 12 in. above the ground line at abutments.

For a culvert 3x2 ft. there are 4.82 yd. of reinforced concrete, 8.75 yd. of plain concrete and 532 lb. of steel; for a culvert 6x5 ft. the quantities are 11.9 yd., 35.9 yd. and 1,385 lb.; for a culvert 10x6½ ft. they are 17.8 yd., 576 yd. and 3,160 lb.

Cargo-Handling Equipment on Italian Freight Steamer

A remarkable equipment for cargo handling has been installed on a large freight steamship, the "Milazzo," recently put in service by the Italian General Navigation Line, operating between New York City and Italian ports. As may be seen in the accompanying views of the vessel, a row of ten elevator shafts extends along each side of the hull, reaching down to the bottom of the hold, while the elevator guides extend a considerable distance above the deck. There is a tunnel fore and aft on each side of the ship in line with the elevator shafts, and double tracks for narrow-gage cars are laid in these tunnels.

The primary purpose of this equipment is for handling coal, grain or similar bulk cargo. Through chutes in the roofs of the tunnels, grain or other bulk freight may be spouted direct into the cars, which are run to the elevators, hoisted, and dumped into the inclined chutes, shown in Fig. 2. The elevators may also be used for handling freight in and out of the hold when the vessel carries general cargo. It is claimed that with this system of cargo handling it is possible to load and unload the ship in a very much shorter time than with the ordinary ship hoisting tackle and thus reduce the delays in port.

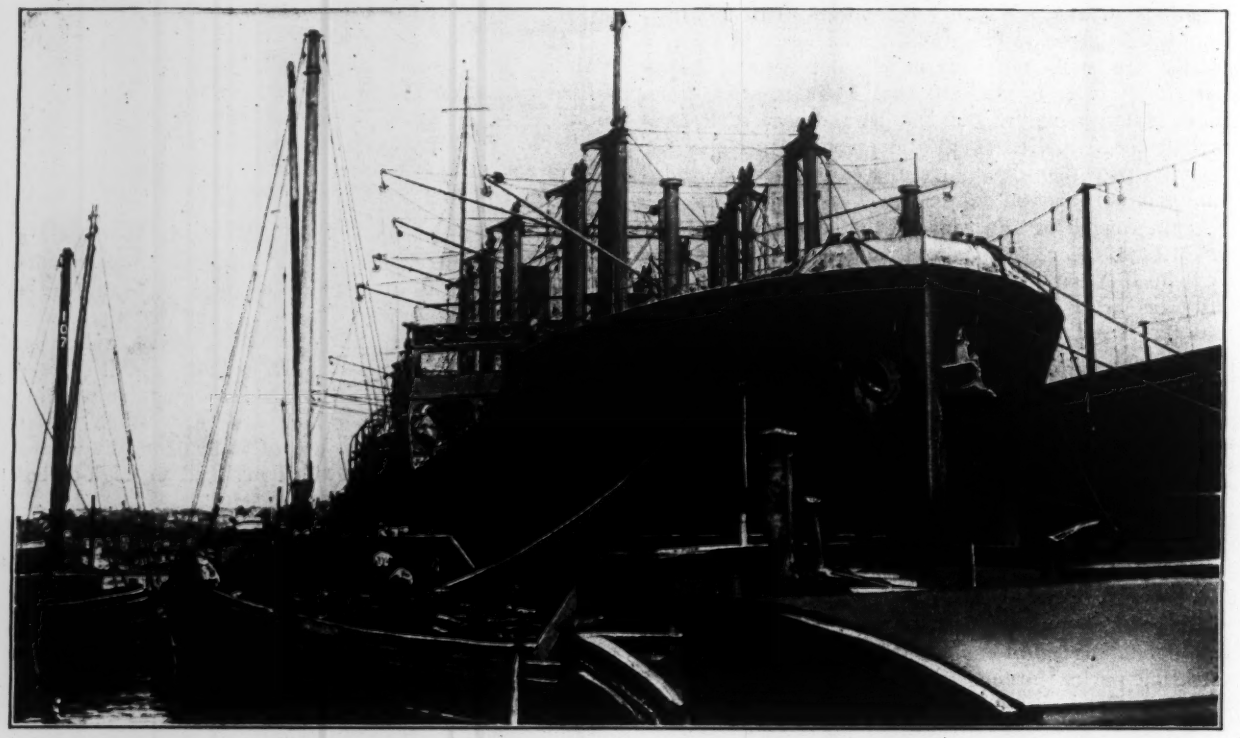

FIG. 1. AN ITALIAN FREIGHT STEAMER, THE "MILAZZO," FITTED WITH CARGO ELEVATORS

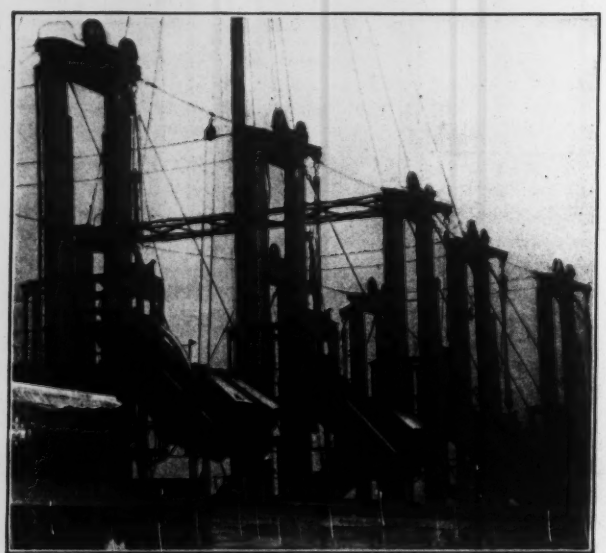

FIG. 2. VIEW OF UPPER WORKS, SHOWING FOLDED CHUTES FOR DISCHARGING BULK FREIGHT

The new vessel was launched in Spezzia, Italy, in May last and is just completing its first round trip across the Atlantic. The vessel is 512 ft. in length, 33.8 ft. in depth and 65.8 ft. beam. She has a displacement of 20,040 tons and is driven by quadruple-expansion engines of 4,000 hp., giving her a speed of 11 knots. A sister ship, the "Volturno," building in the same yards, will be put in service in the fall.

Litigation Over Paving Contracts at Nashville, Tenn., between the city authorities and the Southern Bitulithic Co. was terminated by a compromise agreement on July 8, under which the city pays the company for the work it had completed up to the time construction was stopped by court injunctions a year ago, and the company is to proceed with the completion of the contracts now in force. While this terminates the litigation between the city and the company, taxpayers' suits against the city to prevent it from carrying out the contracts for paving on the ground of their invalidity are still pending. An unusual feature of this complicated litigation at Nashville was the issuance of a court order placing the city's affairs in the hands of a receiver. Before the order was put into effect, however, it was appealed to a higher court, which immediately annulled the order.

Testing Various Soils for Drainage Properties

By John R. Haswell*

SYNOPSIS—The success of a land-drainage enterprise will be greatly influenced by the character of the soil not only as regards its fertility, but as regards its ability to yield up its moisture. The characteristics of different soils which make them readily drained, on the one hand, and difficult of draining, on the other, are here explained, together with experiments to determine the relation between soil textures and freedom of drainage.

There are numerous cases where engineers in general practice can take up drainage work as a side line. A knowledge of soil conditions is absolutely essential, however, to successful designing. An examination of the soil maps published by the United States Department of Agriculture will be found very helpful in this work. Soils are classified according to the diameter of the grains composing them. The standard classification is as follows:

	Millimeters
Fine gravel	2 to 1.0
Coarse sand	1 to 0.5
Medium sand	0.5 to 0.25
Fine sand	0.25 to 0.10
Very fine sand	0.10 to 0.05
Silt	0.05 to 0.005
Clay	0.005 to 0

Loams contain various proportions of the basic constituents and have none of the characteristics of any class. It should also be understood that the name silt merely denotes soil whose particles range in size between a sand and clay and does not allude to the popular idea that it is solely water-borne soil. It is true, however, that alluvial deposits are usually made up of a large proportion of this constituent, because the heavier particles of sand settle in or near the river channel and the clay continues to muddy the water after the silt has been deposited.

It is impracticable to get an analysis of all the soils encountered, but the texture can be determined by placing a small piece of average soil between the teeth. In some soils this makes the sand more perceptible, while the clay content is known by the smooth feeling when the teeth penetrate it. After experience on soils of known classification it is comparatively an easy matter to determine the status of a new soil.

Soils that are formed in the same general way are placed in a soil series. The soils in a series may exhibit all the differences in texture from gravel to clay; and when such is the case, the series is said to be complete. In the United States a series is named from a town or physical feature near where it was first encountered. Take, for instance, two soils in the same class—Sassafras silt loam and Elkton silt loam. The mechanical analysis for texture shows similar classification, but they differ materially in plant production. The Sassafras contains organic matter and drains more readily than the Elkton silt loam, which has been developed under poor drainage conditions and thus has a different structure from the other. It is therefore advisable to know the series to which a soil belongs as well as the class or type, and the color of the soil is of great assistance in this determination.

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Soil Aëration

Practically all the benefits from land drainage are dependent on air entering the soil as the gravitational water passes out, the capillary water being held for the use of the plants. The gravitational water is the surplus water that runs into the drains.

This percolation generally takes place readily in a sandy soil, but in the heavy, tight soils usually classed as clay or as silt loams the circulation of soil water and air is not between the particles, but through the pore spaces separating the granules of soil, the granules being composed of a number of soil grains or particles stuck together. The ideal condition of granulation is intermediate between a compact puddled mass of tightly drawn soil grains and the rough clods that the surface of such a soil breaks into when plowed. The first will not support plant growth, and the latter is hard to cultivate, so besides constituting the ideal from a drainage standpoint, this granular or crumb structure, as it is sometimes called, is the desire of the farmer. The granule best adapted to plant growth varies in size from about 0.5 to 0.01 in. in diameter. Farm crops demand soil air for their growth, and this is present between the granules.

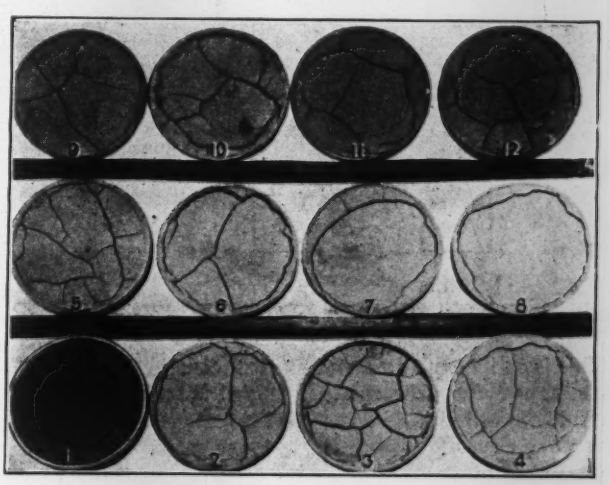


FIG. 1. VIEW OF PUDDLE-TEST SOIL SAMPLES AFTER DRYING

In the experiments outlined the soils used were not absolute separates, but natural soil materials in which predominated those properties of a pure soil separate whose name they bear. Table 1 gives the general proportions of the different classes composing the soils used.

TABLE 1. PERCENTAGES OF SOIL SEPARATES IN SOILS TESTED

Conventional Name	Fine Gravel	Coarse Sand	Medium Sand	Fine Sand	Very Fine Sand	Silt	Clay
Clay	0	2	3	6	9	38	42
Clay loam	2	4	4	12	10	32	36
Silt loam	1	2	3	6	11	56	21
Sandy loam	4	10	12	26	12	22	14
Sand	4	14	20	33	13	9	7

Enough clay to fill three ordinary pie tins was mixed with water until thoroughly puddled. One pan was kept moist throughout the whole experiment, and the other two were allowed to dry. These two were then thoroughly moistened by gentle sprinkling and the process repeated until they had been dried five times, the first pan being allowed to finish with the last two.

The pans that had been repeatedly dried and wet showed the smaller granules and a more even granulation.

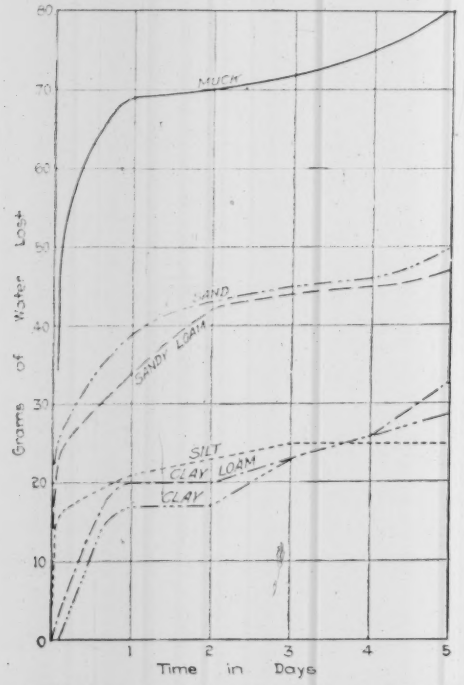


FIG. 2. CURVES SHOWING RELATION OF TEXTURE TO WATER-HOLDING CAPACITY IN VARIOUS SOILS

This is easily explained, since water tends to float the soil grains and thus deflocculate the mass. While in this loose, muddy condition, with no particle of soil attached to another except by being in the same mass, the soil was placed in the pans. During the evaporation of the water the moisture film around the particles had to contract, which produced considerable pressure due to surface tension. The particles were drawn together in small masses or crumbs by repeated drying. Thus the large mass was broken up into crumbs, when fissures or lines of weakness developed. It should be mentioned that the contrary is true in a soil that is too wet, for when a granular soil is kept saturated a long time the granules break up and the soil "runs together," assuming the single-grain structure, where each soil particle or grain is independent of the others and not grouped in a crumb.

To demonstrate the effect of organic matter and sand on the granulation of clay an experiment similar to the foregoing was conducted using straight clay, and clay mixed with half its volume of sand in one pan, and muck, in the same proportion, in another. These pans were wet and dried five times, as previously described. The tin with muck and clay in it showed the finest and most complete granulation, due to the large expansion and contraction of the muck upon wetting and drying. The clay sample ranked next in the uniformity and extent of granulation and also shrinkage of the mass, while the sand mixture showed the least change. The sand particle is inert, and such a soil readily assumes the single-grain structure.

The water in the soil is contained in films around the soil particles and in the capillary spaces of contact between the soil grains. The finer the soil the greater will be the amount of water held, as brought out in a later experiment. In a coarse soil the film around the grain is important, but in the finer soils the water held by capillarity between the grains has by far the larger volume.

Puddle Test

Batches of soil were mixed as given in Table 2, the figures being percentage of total weight of dry soil mixture. After thoroughly mixing, the 12 batches were puddled into tin plates, as in the first experiment, and allowed to dry. Photographs (Fig. 1) were taken to show the extent of the checking, and a relative determination of hardness was made by observing the penetration of a knife-edge.

As would naturally be supposed, there was no checking in the pure sand, sample No. 1 (Fig. 1). Of the others, samples Nos. 7 and 8 showed the least cracking, while No. 3 broke up the most. The first two were the mixtures containing the least clay, while the latter was pure clay.

Arranged in order of hardness, the test showed the following: Nos. 4, 9, 6, 7, 5, 11, 3, 10, 12, 8, 2 and 1. As

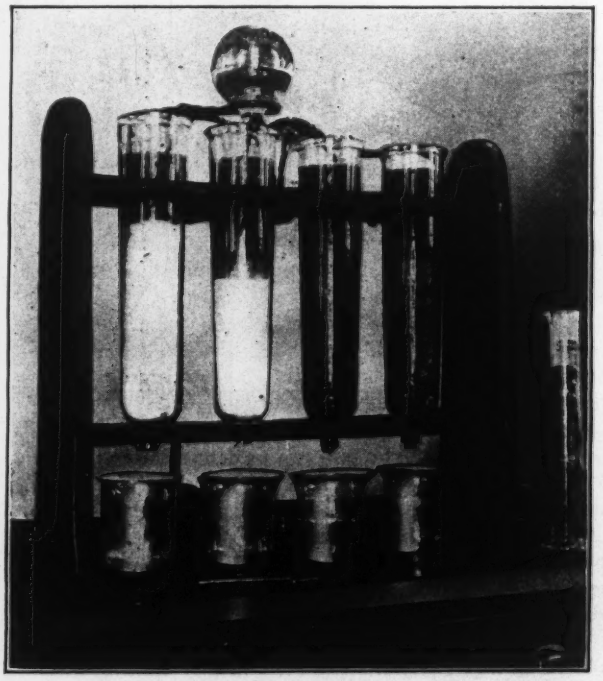


FIG. 3. VIEW OF APPARATUS USED IN TESTS OF RATES OF PERCOLATION IN VARIOUS SOILS

shown by the table, the mixtures containing the largest percentage of sand were the hardest, while of course the pure sand with no cementing material to hold the grains together was the softest. The tests were not fine enough to draw close distinctions between the samples in the middle of the series. Half of the mixtures between the two extremes were about of the same hardness.

The puddle experiment opens up a wide field for thought. The principal idea, from a drainage standpoint, is an indication of how porous a puddled soil will become after it is drained and allowed to dry. In producing a

sand-clay road a hard, nonshrinking surface is desired, and similar experiments should aid in determining proper proportions. Of course, the impact of traffic and washing

TABLE 2. SOIL MIXTURE IN PUDDLE TESTS

No. of Sample	Percentages of Soil		
	Sand	Silt	Clay
1	100	0	0
2	0	100	0
3	0	0	100
4	50	25	25
5	25	50	25
6	25	25	50
7	50	40	10
8	40	50	10
9	50	10	40
10	10	40	50
11	40	35	25
12	10	20	70

of rains would qualify the laboratory findings. Better results would be obtained from a series of wettings and dryings such as would occur in nature.

Influence of Texture Upon Water-Holding Capacity

"The term 'texture' is used to refer to the size of the individual particles of which a soil is composed. . . . The relative proportions of these various-sized particles influence greatly the physical properties of the soil."[1] The amount of water which a soil can absorb and retain against the action of gravity is dependent on its texture, structure and the amount of organic matter contained in it. The first mentioned was the basis of this experiment.

Brass cylinders 1⅞ in. in diameter and 9½ in. long were used in this work. A perforated bottom, covered by a cloth dam, prevented any fine material from seeping through from the tube after being filled with soil and stood in water. A cylinder was used for each of the following soils: Muck, clay, clay loam, silt, sandy loam and sand. The filled tubes were compacted until there was no settlement when the cylinder was dropped 4 in., the final level in all being 1 in. below the top.

The weights of the cylinders having been read before and after filling, the cylinders were stood in water the surface of which was kept at the same elevation as the soil in the tubes. The columns of soil were so short that there was practically no settlement. The time required for free moisture to appear on the surface of the soil was recorded as far as possible.

The cylinders and saturated soils were next weighed, no water being allowed to escape from the scale pan. They were then covered with water glasses and weighed at the end of one hour and each day for five days. The percentages by weight of water taken up, water held and water lost, at the end of the fifth day, are given in Table 3.

It will be noticed that the water appeared at the top of the tubes in the following order: Muck, sand, sandy loam, clay loam, with clay and silt over night and therefore indeterminate.

TABLE 3. TEXTURE AND WATER-HOLDING CAPACITY

Soil	Time Required for Water To Appear		% of Water by Weight			
	Hr.	Min.	Taken Up (At End of Fifth Day)	Lost	Retained	Lost First Day
Clay	Over night		39.5	5.7	33.8	3.3
Clay loam	7	40	40.7	6.8	33.9	4.1
Silt	Over night		37.3	4.6	32.7	2.9
Sandy loam	1	30	31.2	8.0	23.2	5.8
Sand	0	40	27.0	7.8	19.2	6.1
Muck	0	25	55.9	28.1	26.8	22.9

The curves in Fig. 2 were plotted from the daily record. A number of runs would have established smoother curves, but the results obtained show up natural soil conditions very well. In general, clay parts with its moisture very slowly and the other soils in approximately the following order: Clay loam, silt, sandy loam, sand and muck. It will be noticed that the silt curve is the flattest. This soil lost all the drainage water in the first three days, after which it held the remainder while the clay continued to lose.

Without the last column in Table 3 this experiment would result in confusion. We often hear that sand "holds more water than clay" and that the "runoff from sandy lands is greater than clays." The table shows that for the same condition of dryness clay will take up more water than sand, but will not part with as much, and that the rate of drainage from the sand is almost two times as fast as from the clay for the first 24 hr. Silt is shown to be the slowest to drain, while the loams come next to the pure sand.

Percolation Experiment

Percolation jars 20 in. long and 4½ in. in diameter were arranged as shown in Fig. 3. A constant head of 1¼ in. was maintained by means of an inverted flask whose neck just touched the water surface in a beaker used as a distributing reservoir. The beaker cannot be seen in the figure, but the distributing siphons leading from it are shown. The water of percolation was received in a beaker placed under each jar.

The soils used were clay, silt, sandy loam and sand, which were tested loose, compacted and with the addition of 1% of lime by weight with the soil compacted. The percolation through silt and clay was so slow that long runs had to be made in order to get fair averages, the results being reduced to grams of water per hour. Reading from left to right in Fig. 3, the soils in the jars are clay, silt, sand and sandy loam from a subsoil.

Tables 4 and 5 show that coarse-textured soils permit the most rapid percolation both in quantity of water passing and in the time required for the water to appear.

TABLE 4. PERCOLATION OF WATER THROUGH COLUMNS OF SOIL

Soil	Grams of Water per Hour — Treatment		
	Loose	Compact	1% Lime, Compact
Clay	13	0.6	1.1
Silt	5	1.2	2.7
Sandy loam	139	9.0	15.0
Sand	269	58.0	25.0

TABLE 5. TIME REQUIRED FOR WATER TO DRIP

Soil	Hours and Minutes — Treatment		
	Loose	Compact	1% Lime, Compact
Clay	Over night	93.00	26.00
Silt	Over night	41.00	13.00
Sandy loam	1.15	5.45	3.30
Sand	0.30	0.35	1.05

The larger the total pore space in the soil the greater the percolation and the less time required for the water to pass. A comparison of the loose and compact columns in the table shows that soil cultivation can be made to assist drainage.

The effect of lime corresponds with actual results obtained on farms. Lime will open up a tight soil by flocculating the soil grains and will make a sandy soil denser for the same reason. The lower percolation in sand after the addition of lime shows this very clearly. The lime shortened the time required for the water to appear in all the soils except sand, which was made to have less pore space due to the action of the lime and hence retarded the passage of the water. This shows that leaching from sandy soils can be lessened by the use of lime.

[1] Lyon and Fippin, "Soils," p. 70.

In considering the drainage possibilities of a tract the following outline of procedure is recommended in reference to the soil:

1. Obtain a soil map of the area, if available, and locate the spot.

2. Determine the uniformity of the soil and see if there are any areas too small to be shown on the map.

3. Observe the series to which the soil belongs and see by reference to the text accompanying the soil map if the members of the series are usually well drained.

4. Determine the classification and examine the subsoil for pervious or impervious strata, joint clay formation, etc.

5. Note any color changes in the soil.

6. Compare, if possible, this soil with some similar soil which has already been drained.

It must be remembered that the change in structure mentioned under "Granulation Experiment" and "Puddle Test" would not occur immediately after draining, but would require several years in a fine soil. This accounts for the increasing yields for the first three years, so often seen in drainage records. After the soil becomes well granulated, the production fails to increase further from drainage.

When a drain is first opened, it is interesting to watch the water-bearing strata discharge into the ditch. After a short time, usually within a week or two, the flow falls off and sometimes ceases until after a rain. Then as time goes on, the discharge from the drain increases in amount and duration after storms, showing the effect of better soil conditions. This is the explanation for the statement that extensive tile drainage will reduce flood flows by changing a saturated, unabsorbing soil into one which will absorb rainfall and liberate it gradually instead of letting it run over the ground surface.

The practical uses to be made of the foregoing data have been set forth by the writer in a published bulletin.[2] A few extracts sum up the conditions governing depth and spacing of underdrains:

Three feet is the common depth for tile in average soils growing the ordinary crops. The depth should be greater for alfalfa and less for strawberries because of the difference in the root systems. . . .

In a tight "post oak" (silty) soil the percolation of water is very slow and of small extent, so that drains but 2 ft. deep are possible, while in open loams drains as deep as 5 ft. have proved the most satisfactory. According to Fippin,[3] up to a depth of 4 ft. an increase in the depth of the drain will permit an increase in the interval between drains of about two rods for each foot in depth. For example, a drain 4 ft. deep will drain a strip two rods wider than will a drain 3 ft. in depth, provided, however, that the line be laid in an open soil. The only extra expense is that of an extra foot of digging, which is more than offset by the smaller number of laterals required. Where the subsoil is a heavy clay overlain by an open topsoil, it is good practice to lay the tile at least as deep as the line of division. Under these conditions the soil water generally spreads over the subsoil and can be reached only by tile laid as suggested.

In the so-called uniform system of drainage the number of laterals can be greatly reduced if the depth is a maximum for the soil in question; for the laterals must be so closely spaced that no unaffected land will lie between them, and the greater the depth of drain the wider the strip of land affected. In tight, close soils the drains may have to be but 2 ft. deep and 40 ft. apart for efficient drainage, while a depth of 4 ft. and a spacing of 200 ft. may produce equally good results in a loose open soil. . . .

The presence of a "joint clay" formation in the subsoil is a great help to under-drainage, as the water will follow the joints. The presence of this type can be detected by digging a test pit; if the subsoil breaks up into small cubes, it is said to be of a "joint" formation.

Some retentive subsoils must be broken by deep plowing before they will permit tile drains to operate freely. A subsoil plow run in the bottom of an ordinary furrow will generally give a total depth of plowing of 18 in. In some sections, after years of cultivation to the same depth, a "plow pan" or "plow sole" of firmly compacted earth has been formed by the tramping of the horses and the downward pressure of the common moldboard plow. This bed of tightly packed soil must be broken before thorough drainage can be accomplished; and even when underdrains are not installed, the good effect of such subsoiling will often be appreciable. Retentive soils are frequently rendered susceptible to drainage by heavy lining. . . .

[2] "Land Drainage in Maryland," Bulletin 186, Maryland Agricultural Experiment Station.

[3] "Land Drainage in New York," by E. O. Fippin; Circular 70, New York Department of Agriculture.

Dished Surface for Concrete on Hilly Country Roads

Sioux City, Iowa, which has gained fame on account of the excellence of its concrete pavements, is extending paved streets so far out of the business and residence section of the city that in much work now under way the conditions are those of typical country roads. These pavements are built under the supervision of T. H. Johnson, City Engineer.

On level stretches or moderate grades pavements are of the ordinary crowned cross-section, 16 ft. wide, 6 in. thick at the sides and 8 in. at the center. But on grades of 4% or more the crown is replaced by a concave, or dished, cross-section. The surface of the pavement is

DISHED CONCRETE PAVEMENT, SIOUX CITY, IOWA

made 3 in. lower on the center line than at the edges, but of uniform thickness (7 in.). The 8-ft. earth shoulders are crowned so that the drainage of half of them goes to the center of the pavement and the other half to the usual side ditches.

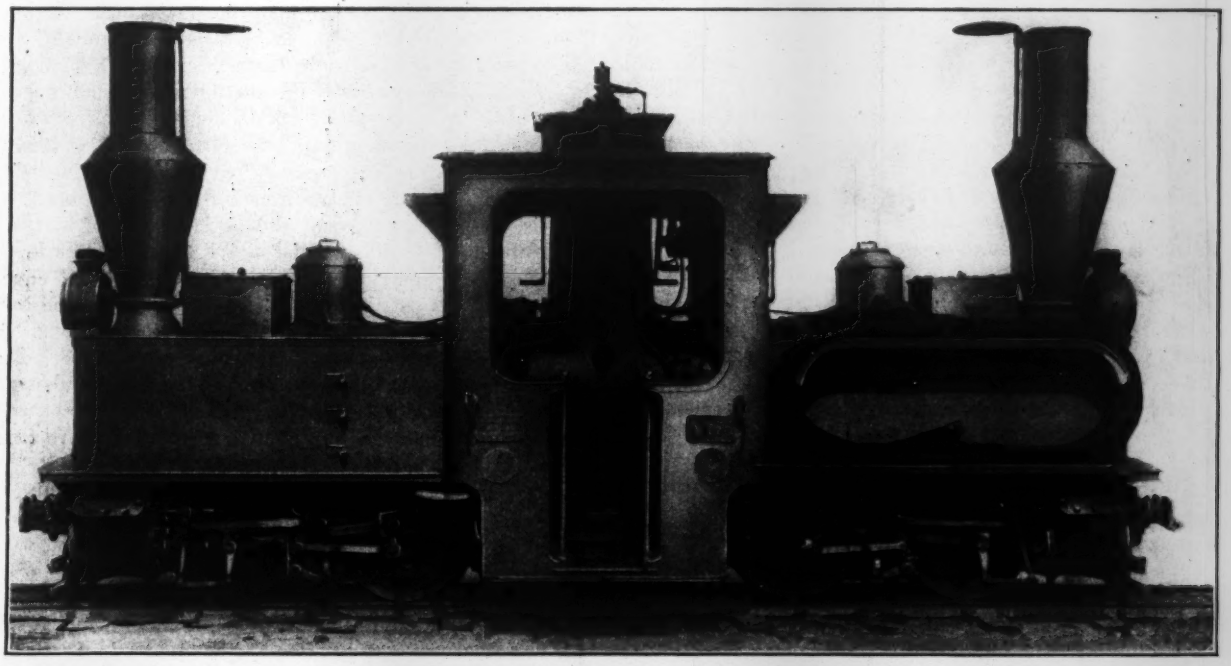
The object is twofold. The pavement takes a large part of the runoff during rains and saves washouts which almost invariably occur on unpaved hill streets with every severe shower. The character of the soil in the hills is very peculiar. It will stand for years in cuts with a perpendicular face, but erodes under running water with astonishing rapidity. The other object is to preserve the shoulders of the pavement. The tendency with a crowned concrete pavement is for the water and traffic to form a gulley along the edge of the pavement slab. With the dished cross-section there is no such tendency. The soil on the shoulders tends to work in and form a cushion to protect the edge of the slab.

The water that runs down the center of the pavement is diverted at the foot of the slope by means of dished concrete aprons across the shoulders to the side ditches.

Beyond the aprons the crowned pavement begins again, the transition being so gradual as to make easy riding, and the change is not conspicuous. The dished pavement of the hill, however, is rather noticeable, and the appearance is not so pleasing as that of a crowned roadway.

French Military Locomotives Built in America

The curious locomotive shown in the accompanying view is one of 280 engines built for the French Government by the Baldwin Locomotive Works, of Philadelphia, and intended for use on military railways of 24-in. gage. They are of what is known as the Pechot type, being named after their inventor. They are flexible-wheelbase noncompound engines of the 0-4-4-0 class. Their general design is similar to that of the earlier Fairlie type, having a double tandem boiler (with two barrels and fireboxes) and being mounted on two steam trucks, so that the engine is powerful for its size and exceedingly flexible. In the Mallet type of engine there is only one steam truck, the second group of driving wheels being carried in fixed frames.



AMERICAN-BUILT FLEXIBLE LOCOMOTIVE OF 24-IN. GAGE FOR THE FRENCH GOVERNMENT
Baldwin Locomotive Works, builder

In these new Pechot locomotives the boilers have 25-in. steel barrels and a steel shell inclosing the inside fireboxes, which are of copper, 16½x23½ in. The tubes are of brass, 1⅝ in. in diameter and 5 ft. 8 in. long. Each boiler barrel contains a separate set of tubes, and there is a smokebox and stack at each end of the locomotive. The dome is placed above the fireboxes. It contains two throttle valves, one serving each truck. Steam is normally admitted to all four cylinders; but if desired, the locomotive can be operated by one steam truck only. The boiler barrels are supported by saddles placed directly over the center pins of the trucks, while the firebox shell is supported by two plate frames that are riveted to the saddles. The boiler simply rests on the saddles, without being attached to them, thus allowing for expansion and contraction.

The four cylinders are all 7x9.6 in. The live-steam pipes leave the boiler through openings in the bottom and pass down through the center pins of the trucks, so that the ball joint in each pipe can be arranged to coincide with the center on which the truck swivels. The steam distribution is controlled by balanced slide valves, driven by Walschaerts valve gear. Reversing is effected by means of a hand lever. The engine has 26-in. wheels, with 3-ft. rigid wheelbase for each truck and a total wheelbase of 12 ft. 6 in. The weight is about 14 tons.

The cab is located at the center, the engineman being on one side and the fireman on the opposite side. The water-supply, 400 gal., is carried in four tanks placed beside the boiler barrels. The two tanks on the fireman's side are shorter than the others, so as to provide space (adjacent to the cab) for coal bunkers of 880 lb. capacity. Each firebox has a side door and is fired independently of the other. The American-made equipment includes injectors, whistle, safety valves, lubricator, steam gage, headlights and cab and signal lamps. The locomotives are fitted with hand screw-brakes and with drawbars of the radial type.

These locomotives were erected at the Eddystone plant of the builders, and were subjected to severe tests, each being run under steam for approximately half a day on a special track built for testing purposes. Each locomotive was shipped complete in one case. The smokestacks and a few small fittings were removed, but were boxed with the locomotive, so that very little erecting work was necessary on arrival at destination. Considering the facts that the locomotives were built throughout to metric measurements and were of most unusual design, quick records for delivery have been made.

River Terminals Projected for St. Louis

The commercial interests in the cities of the Mississippi Valley which have for years been urging Government appropriations for waterway improvement have come to understand that even more important to the revival of waterway traffic than improved channels are improved terminal facilities at the principal cities. The most important port on an inland river in the United States is St. Louis. The tonnage of water-borne traffic at Pittsburgh, it is true, is much larger, but the Pittsburgh water traffic is almost wholly of coal, while the St. Louis water traffic is general merchandise of much higher value per ton.

As is well known, there has been a great falling off in the traffic on the Mississippi River. In 1887, the freight shipments by river to and from St. Louis amounted to 1,290,000 tons. In 1914 the river traffic had dwindled to 154,000 tons. In contrast with this, the total rail shipments at St. Louis in 1914 amounted to over 52,000,000 tons and the freight taken by rail lines approximately parallel to the rivers was about 28,000,000 tons. Notwithstanding these impressive figures, there is a large public sentiment in the Mississippi Valley which favors the creation of facilities that will make the revival of river traffic possible, and this sentiment is as strong in St. Louis as anywhere.

A report on a proposed terminal system for river traffic in St. Louis was recently made by S. W. Bowen, Chief Engineer of the City Department of Public Utilities. The report is a voluminous document, containing with its appendices a great amount of statistical matter, and extensive maps and drawings illustrating the structures it is proposed to build.

Mr. Bowen first investigated the question, What amount of river traffic should the proposed terminals be planned for? A canvass was made of the principal manufacturing and shipping concerns of the city, which resulted in tentative promises of about 250,000 tons of freight annually for shipment by river. While this would be a material increase over the present river traffic it is noteworthy that out of 350 letters sent out only 114 replies were received. Mr. Bowen thinks it probable that if the proper terminal facilities were once furnished, a considerably larger tonnage of river traffic than that stated above might be reached, probably as much as 500,000 tons. Even this traffic would not be sufficient to warrant the city in making a very large outlay on river terminals. Mr. Bowen believes, however, that there is opportunity for considerable traffic to develop between the Pacific Coast ports and Mississippi Valley points by way of the Panama Canal and New Orleans. There will also be a further increase in the territory opened to river traffic from St. Louis if the proposed Intercoastal Canal along the Gulf is completed.

The class of freight most likely to be handled on the Mississippi River in the future appears to be general merchandise, except that some grain may possibly be handled by river, if the business were of sufficient volume to justify the building of grain elevators at river terminals for delivering material to boats. Mr. Bowen outlines the requirements for an efficient terminal system for inland river traffic as follows:

> An efficient river terminal system should consist of a series of docks conveniently located with respect to the manufacturing and distributing centers, and a complete railway system connecting the docks with all of the railways entering the city. The docks should be fully equipped with the most modern freight-handling machinery. There should be warehouses on the docks for the protection of transient freight and storehouses lying back of the warehouses for freight to be held some time. The railway system should include a belt line with receiving and storing yards and small sorting yards at each dock.

St. Louis is better able than most river cities to construct a river terminal system since 50% of the total river frontage is owned by the city. The city also owns a single-track railway line about 7 mi. long extending along the river front in the northern part of the city. In addition, part of the St. Louis Transfer Ry. system came into the city's possession on Dec. 15, 1915. It is proposed to extend these railway lines so as to form a belt line entirely around the city, with connections for the transfer of cars at each railway intersected.

There are a number of suitable locations for municipal docks along the river front, and local conditions will determine which of these sites are preferable for first development. In the design of the docks, the Government requirements are that the river bed in front of the docks shall be dredged to a depth of at least 8 ft. below low-water mark and shall be maintained at that depth. Fills must be made above and below the dock to form easy approaches and prevent the formation of eddies.

The elevation of the roadway paving on the dock site is fixed at the same elevation as the high-water mark of the record-breaking flood of 1903. The floors of wharves and warehouses would be placed 3.7 ft. above this level. Mr. Bowen thinks it likely that if another flood equal to that of 1903 should come, after the levees on the east side of the river are built, the high-water mark at St. Louis would be raised 2 to 3 ft. above that of 1903. The floors of warehouses and wharves would still be above a flood of such a height, and the small depth of flood water would do no damage to the paving for the short time it might flow over it.

After an extensive study of freight-handling machinery suitable for use at docks, the decision was in favor of locomotive cranes in connection with motor trucks, with the possible later addition of gantry jib cranes. With such equipment, it is estimated that miscellaneous package freight can be handled for about 20c. per ton. The present river traffic at St. Louis is handled by four packet lines and three tow-boat lines. It is estimated that to provide for present requirements, with a reasonable provision for future growth, four 300-ft. docks would be required on Valentine St. and the same number at North Market St.

The estimated cost of these docks and warehouses, with their mechanical equipment, is $1,230,000. For extensions of the municipal railway system, about $554,000 would be required. Adding a reasonable percentage for contingencies, the total estimated cost of the whole municipal system would be $2,000,000 in round numbers. At North Market St., the dock construction on the lines proposed would reclaim about 17 acres of land. It is estimated that this could be sold for enough to reduce the net cost of the project to $1,500,000.

In estimating the capacity of these docks to handle traffic, Mr. Bowen assumes an annual freight transfer amounting to 550 tons per lin.ft. of dock, which corresponds with European experience in mechanical cargo handling. At this rate, the 2,400 lin.ft. of docks pro-

vided would be capable of handling 1,320,000 tons of river traffic. The annual expense to the city of this proposed dock and railway system, including interest at 4%, sinking fund at 2.06%, and maintenance, is estimated at about $208,000. To provide a revenue to meet this expense, Mr. Bowen recommends that a rate of 10c. per ton be charged for freight handled over the docks, and for the use of the municipal belt line that a rate per car be fixed or else that the line be leased at an annual rental.

Causes of Rail Corrugation

The corrugation of rails on street railways, with its causes and the means to prevent and remove it, has been the subject of extended discussion. The following statement as to the causes, which are listed in the order of their importance, has been prepared by Mason D. Pratt, Chief Engineer of the United Railways and Electric Co., of Baltimore, Md. It is based on twenty-five years of experience in the manufacture of rails and the construction and maintenance of track. He states that this was submitted to the chief engineers of all the larger street-railway systems in the country, and that while their individual experience varied somewhat, the conclusions were substantially agreed to by those who had given the matter careful consideration. The causes are as follows:

1. Self-propelled cars. Practically all corrugation occurs on electric railways where all or a large majority of the cars are self-propelled.

2. Girder or other rails having considerable vertical stiffness.

3. Soft steel for rails. The extent and depth of corrugation is in inverse ratio to the content of carbon and other hardening elements.

4. Single type of cars in operation; particularly double-truck, four-motor equipment. On tracks over which there is regularly operated a service of mixed types of cars, corrugation may appear from time to time, but unless some other feature (as for instance No. 3) is the cause, the corrugation will usually "iron out."

5. Contour of rail head. Flat-headed rails, or those on which the wheels have only a narrow or partial bearing, will develop corrugations earlier than those on which the wheels have a full bearing (as on convex-head rails).

6. Heavy mill scale on the rails. On rails that have been rolled under conditions producing heavy mill scale that is allowed to go through the finishing pass, the surface of the rails has been so heavily marked as to start corrugation immediately after the rails have been put in service. Mr. Pratt has inserted the following clauses in his specifications for rails for the last two years, and this has had a beneficial effect.

In the rolling process such precaution shall be taken as may be necessary to entirely remove mill scale just prior to rails entering last two finishing passes and otherwise to produce a finished bar having smooth, regular and true surface on head and in finishing contact areas.

Any rails showing by the closest possible inspection the slightest indication of corrugation will be rejected.

Other causes are: (7) Rapid acceleration and deceleration. (8) Unequal diameter of wheels on same axle. (9) Long radius curves. (10) Intensity of wheel loads. (11) Track foundation. (12) Segregation of steel. (13) Track sanding. (14) Grades.

Austin Dam Section Criticized

That the novel and radical section of the new part of the Austin, Tex., dam has less stability than the originally planned hollow section, is the statement of Bartlett & Ranney, Consulting Engineers of San Antonio, who were recently employed by the city to report on the dam. As noted in the article in *Engineering News*, June 3, 1915, p. 1089, the width of the base of the hollow reinforced-concrete dam built across the Colorado River to fill in the spaces left when the masonry dam there was washed out in 1900 was reduced materially by carrying the downstream deck about 20 ft. upstream of the top of the upstream slanting deck, in distinction to the usual hollow-dam section in which the two decks meet at a peak. The original hollow section designed for the site had a base of 125 ft. 9 in. The modified section has a base of 93 ft. The change was made by the contractor with the approval of the engineer under a clause in the specifications which provided that the contractor might make any change that, in the opinion of the engineer in charge for the promoting company, would be desirable, provided such changes did not decrease the amount of equipment in the power house, the height of the dam or its stability, the size of the reservoir, or in any way decrease the capacity or completeness of the installation or in any way lower the standard of the grade of construction.

The consulting engineers' report regarding this section is as follows:

Considering the structure as a whole in relation to its stability on the foundation—a matter of fundamental importance and especially so in view of the notoriously faulty character of the foundation—we find that the change in design did decrease the stability. But we find that as respects the rigidity of the structure itself and the strength of the slabs which support the water, the adopted design is superior to the original design. The character of the foundation at the site of this dam, the failure of the old dam, the many investigations and studies which have been made in connection with the failure and proposed rebuilding, the certainty of a considerable silt pressure in time, as well as the unusual height of the structure for an overflow dam, all indicate the emphatic need for extreme conservatism in reconstruction.

The sliding tendency of 0.51 for the contract dam is increased to 0.58 in the dam as constructed. Under the conditions, we do not think that any increase in the sliding tendency is justifiable.

In general, we are forced to state that as respects its fundamental stability on its foundation, the dam as rebuilt is a radical design while the dam as designed was of conservative type.

The consulting engineers also reported that while not embraced under the scope of the report as pertaining to the changes in the original contract and specification, they considered that the old masonry portion of the dam, which is now the middle stretch of the present dam, has an insufficient factor of safety against sliding.

Workmen's Compensation to be paid the dependents of 290 workers killed in industrial accidents in Pennsylvania during the first five months of 1916 aggregate $742,962.75, according to a compilation made by the State Bureau of Workmen's Compensation for Labor Commissioner John Price Jackson. During this period 862 workers were killed; and it is estimated that when all the compensation agreements are perfected, the total amount for fatalities will approximate $1,500,000. The average total payment in each of the 290 cases is $2,560.90, and the average weekly wage of the 862 killed was $14.86, or a total annual payroll of $666,084.64. The majority of the fatalities occurred in mines. The division by occupations is: Miners, 375; laborers, 187; iron and steel workers, 157; railroad employees, 119; clerks, 7; firemen, 9; salesmen, 5; civil engineers, 1; policemen, 1; motormen, 1; teachers, 1; carpenters, 1.

Field and Office

Drafting-Room Practice in a Departmentalized Shop

Five distinctive features individualize the drafting room of the Wellman-Seaver-Morgan Co., Cleveland, Ohio. They prove that success in the drafting department is by no means incompatible with free departure from conventional methods to fit special needs of the business or personalities.

Leading features are: (1) Departmentalization of the drafting room to match the departments of the engineering organization; (2) functionalized control, in which the engineering direction and administration are separated; (3) cost recording applied to the single drawing, with a card file for all drawings; (4) a system for limiting house-account jobs; (5) use of cut sheets.

The company builds a large range of structures and machines, and its engineering work covers at least two broad classes of structural steel and three classes of mechanical-engineering construction. This condition is met by a complete departmentalization of the engineering branch, in fact of the whole company except the shop. In many concerns where this is the case, the drafting room is a single unit, technically and administratively, but here the drafting room also is departmentalized. Long use of this plan of operation proves its success.

The six departments, each headed by an engineer, are: Coal and Ore Handling, Hydraulic Turbine, Mine Hoist, Crane (charging machines and coke-oven equipment), Furnace and Estimating. The drafting force is divided into corresponding groups, and each group is headed by a foreman. Including an extra group, for billing and orders (3 stenographers and 3 clerks), the whole force comprises normally 90 men. The 44-hr. week is used.

FIG. 1. CORNER OF DRAFTING ROOM

Administrative control of the drafting force is in the hands of the Engineer of Construction, who employs, disciplines and manages. For technical instructions, however, the group foremen are directly under orders of the department engineers. There is no one corresponding to the Chief Draftsman of the usual type of organization. Finished drawings are checked by the group foreman and are countersigned by the department engineer.

This division of functions is not merely formal but is applied consistently. Thus, space limitations made it necessary to locate two groups of the drafting force away

FIG. 2. DRAFTING TABLE OF OWN DESIGN

from the main drafting room—in fact directly under the eye of the respective department heads. These groups nevertheless are under the same administrative control as the force in the main room.

In this organization good disciplinary control is secured without a time clock. The group foremen are responsible for their men to the Engineer of Construction. They are accountable for both regularity and order, in addition to efficiency.

Record Forms and File Cards

To obtain a permanent record of drawings and a reliable basis for estimates of drafting costs an unusually careful system of recording the drafting-room operations was devised. It is based on the draftsman's time card (Fig. 3), which states not merely the job number but also the operation (designing, detailing, tracing) and the drawing number. The group foreman countersigns and turns in the time cards of all his men. The original goes to the paymaster; a duplicate is filed in the drafting room.

On completion of a drawing, it is turned in to the Engineer of Construction, accompanied by a report card (Fig. 5) giving all pertinent data that describe the sheet.

DAILY TIME CARD. DRAFTING DEPARTMENT.

THE WELLMAN-SEAVER-MORGAN CO.

NAME _____ TIMEKEEPER'S CHECK _____

DATE _____

CHARGE NO.	OPERATION.	DRAWING NO.	HOURS

APPROVED _____ TOTAL TIME _____

44 11—15 50M

FIG. 3. TIME SLIP GIVES OPERATION AND SHEET NUMBER

except the time spent on it. This report card is filed as a permanent record of new drawings made on a given order.

A cost card for the drawing is made up from the time slips, posted up each day, and totaled when the report card announces its completion. This card (Fig. 6) gives the total time and cost, and an analysis of this time by men and by dates. An analysis by operations can be deduced, when needed, from the rates of pay shown on the card.

Regular (contract) jobs are started by a group foreman on receiving from the Engineer of Construction a duplicate of the order made out by the contracting agent or engineer. This order is returned after completion of all the drawings of the job.

House-account jobs, or what might be called nonproducing work in the drafting room, are initiated by a

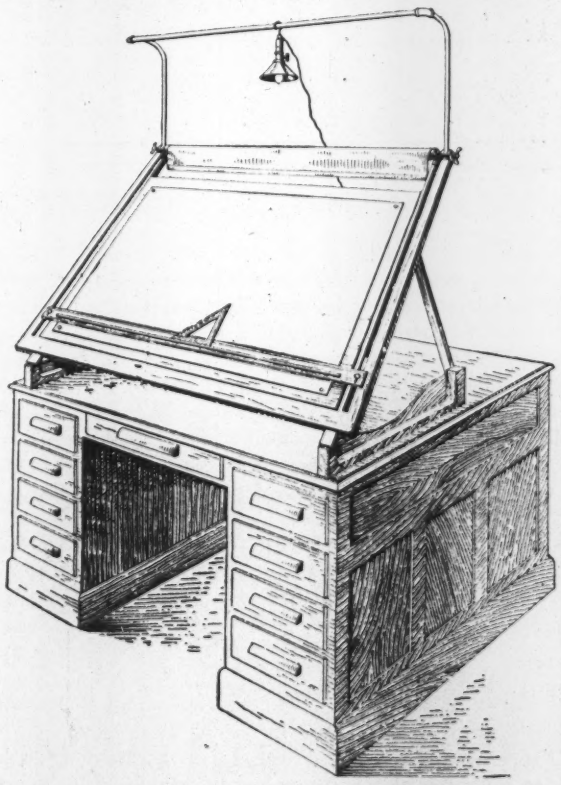


FIG. 4. SKETCH OF UPRIGHT BOARD AND LAMP MOUNTING AS FORMERLY USED
Now abandoned for horizontal tables and ceiling drops

THE WELLMAN-SEAVER-MORGAN CO.

CONTRACT No. _____ PROPOSAL No. _____

SUBJECT _____

_____

DRAWN BY _____ DATE _____

CHECKED BY _____

APPROVED BY _____

TRACED FROM _____

FOR _____

SCALE _____

FIG. 5. REPORT CARD FOR INDIVIDUAL DRAWING

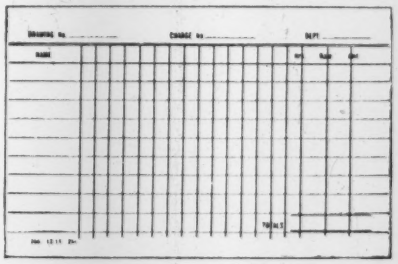


FIG. 6. COST RECORD CARD IS FILED FOR EACH SHEET

special order made out on a numbered-series card issued by the Engineer of Construction. This card, besides tending to limit the amount of such work, helps to ensure proper locating of the charge in the accounting department. The latter result is secured by sending a duplicate of the card to the accounting department when the job is initiated, and giving on this card a sufficient description of the proposed work that the latter department is enabled to place the charge against the proper one of several standing accounts authorized by the management of the company. Fig. 7 shows the card and its duplicate. The back of the duplicate is ruled for the daily time and cost figures from the time slips, the time-slip entries being identified by the number taken from the special-order card.

Equipment of the Drafting Room

The arrangement of the room is determined partly by the space layout and the window lighting. The room is about 40x100 ft., and 15 ft. high; about three-fourths of the wall space has windows. With narrow piers, and windows extending to the ceiling, the side light easily allows of placing two tables in line from the wall. As the column arrangement makes an 8-ft. aisle down the middle of the room, the row of two tables on either side just fills the available space. In the other direction the tables are spaced about 4 ft. 8 in., or three per 14-ft. bay.

Present practice as to equipment is of relatively recent date. Up to eight years ago the room was fitted with upright drawing boards mounted on easel brackets set on top of office-type desks. Fig. 4 sketches roughly the desk and board, and its lamp fitting. The brackets could be adjusted to different angles, the board (about 32x40 in.) could be shifted up and down on the brackets and could even be slid down to rest on the draftsman's knees, and a parallel-ruler working in slots of the rails served in place of the T-square. The lower stile had a projecting tray to hold pencils and tools. A pipe support

over the top carried a baby Nernst lamp whose hook could be slid along the pipe rail to any position.

The horizontal-board plan is now used exclusively. Many of the old desks were adapted for this by putting 6-in. legs under them; two desks with a filing case between give enough table length for two draftsmen. The rest of the room is equipped with large tables designed and built by the company. Fig. 2 shows one of these quite clearly. There are two drawers for tools and books, and two large ones for drawings. The top is 34x84 in.; it has an oak border and a soft-pine filling. This is

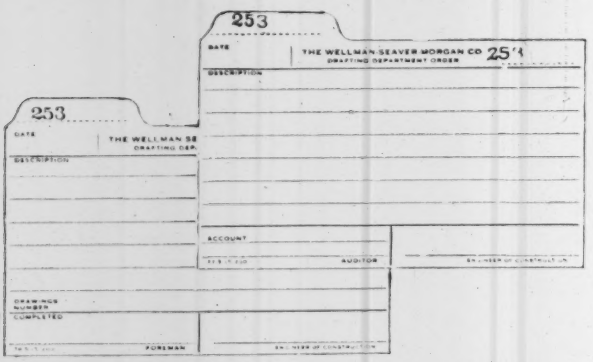


FIG. 7. DRAFTING-ROOM ORDER CARDS FOR HOUSE-ACCOUNT JOBS

Cards 3x5. Lower, white, goes to drafting-squad foreman as his order. Upper, buff, goes to auditor, and is ruled on back for time and amount

rarely or never used for pinning down drawings, but a drawing-board on taper rails is placed on it.

Several large boards are used in the Hydraulic Department. These range from 4x6 to 5x10 ft., and have a steel edge for the T-square. For other departments the regular table serves satisfactorily.

There are a few drafting machines, but most of the tables are equipped with T-squares. Triangles and other

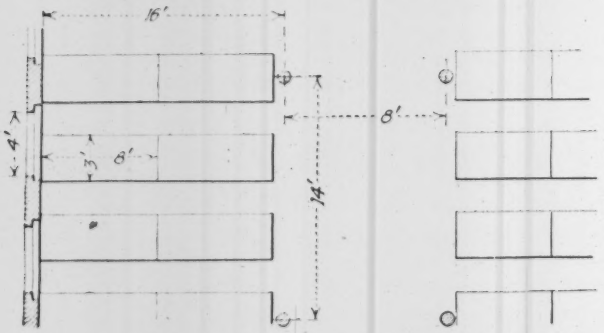


FIG. 8. PART PLAN SHOWING TABLE ARRANGEMENT

tools are furnished by the men. Special tools such as pantograph, beam compasses and special curve and radius templets are furnished by the company.

The present lighting system has one 360-watt drop fixture in each bay (14x16 and 8x14 ft.). The fixture has six 60-watt tungsten lamps placed under a 39-in. General Electric "diffuser," and inclosed below by a pearl-glass shade, all hung about 6 ft. from the ceiling (9 ft. from the floor). The illumination is said to be entirely adequate. Ceiling and walls, painted buff in oil, are important factors in the lighting.

Most of the work is done on cut sheets. Buff drawing paper is used for pencil layout and detailing, and this as well as the tracing cloth and blueprint paper is bought in sheets. The tracing cloth is bought with border and company stamp (lower right corner) printed on.

The reason for using blueprint sheets is that the two machines—imported vertical machines, among the first made—take sheets only.

There are four standard sizes of drawing sheet; only in turbine work and other special cases is it necessary to depart from these. They are:

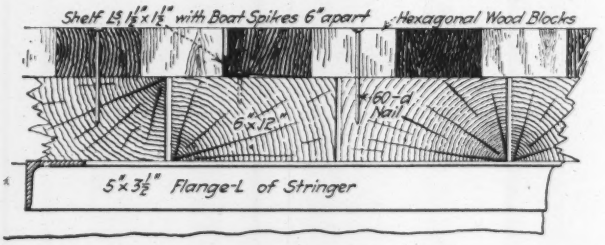
Over All	Inside Inner Border Line
26 x36 in.	23¾ x33¼ in.
18 x26 in.	16 x23½ in.
13 x16 in.	10¼ x14 in.
9½ x12 in.	7¼ x10⅝ in.

The sheet sizes are selected by the group foreman in planning out the work on a particular job. The smallest size is used for drawings that are likely to be wanted in letter size for filing with correspondence and notes.

New Wood Floor Construction for Bascule Bridges

For the roadway paving of bascule bridges at Chicago, the Department of Public Works is now using hexagonal wood blocks 3½ in. thick, instead of 4-in. rectangular blocks. One of the main reasons for the use of the hexagonal blocks is that being all heartwood the expansion is radial instead of longitudinal, so that there is less trouble from expansion and contraction movements.

The floor construction used with these blocks makes a very substantial deck, and is shown in the accompany-



WOOD PAVING WITH 3½-IN. HEXAGONAL BLOCKS FOR BASCULE BRIDGES AT CHICAGO

ing drawing as applied to the new Belmont Ave. bridge. There is a course of transverse timbers 6x12 in., laid close together (with ¼-in. spaces) upon the steel stringers. On this planking are placed the 3½-in. hexagonal paving blocks, which are cut from 6- or 7-in. tamarack posts. These blocks are of what is known as the Hex type, manufactured by the Jennison-Wright Co., of Toledo, Ohio. They are treated with 12 lb. of creosote oil per cubic foot of timber. For use on bridges they vary in thickness from 2½ in. to 3½ in. according to the traffic conditions.

On the bascule bridges, each third row of blocks has every block fastened by a single 60-penny nail driven through its center and into the subplanking. To prevent slipping of the blocks due to the raising of the bridge, galvanized-steel shelf angles 1½x1½ in. are laid across the planking at intervals of 6 ft. and spiked in place. The blocks of the adjacent row are cut to an incline on the bottom so as to fit over the leg of the angle.

The paving originally designed for the Belmont Ave. bridge consisted of transverse 4x6-in. planks, 6 in. apart, laid on the stringers and covered with 2x8-in. longitudinal planks laid close together. On these was to be

a layer of tarred paper upon which 4-in. rectangular blocks were to be placed. The blocks were to be fastened by toe-nailing with a 10-penny nail in every block of each third row.

The new floor construction is slightly heavier and more expensive. But it will require much less maintenance owing to its greater rigidity under the heavy motor-truck loading carried by the Chicago city bridges, and to the elimination of troubles from expansion and contraction of the paving. This construction is to be used on all the new bridges, and is being applied rapidly to existing bridges to replace the old plank and Shuman paving. This latter consists of built-up sections formed of planks on edge, bolted together.

The new type of paving has been designed under the direction of John Ericson, City Engineer, and Thomas G. Pihlfeldt, Engineer of Bridges.

Raising Settled Bent of Heavy Concrete Viaduct

Soon after the construction of the western section of the Colfax-Larimer viaduct in Denver, Colo., described in *Engineering News*, Jan. 27, 1916, p. 168, water from

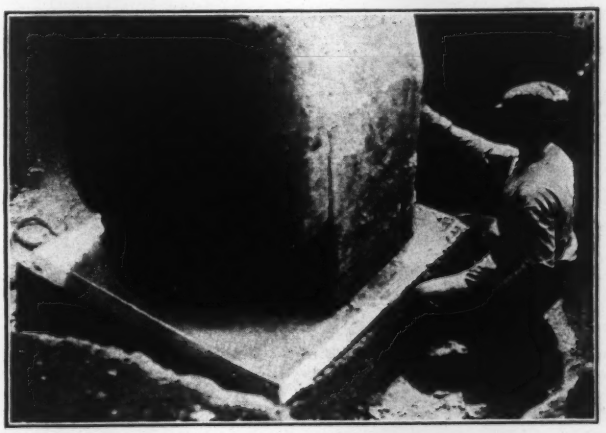


FIG. 1. CLIPPING AWAY CORNERS TO RELEASE BONDING RODS

a leaking hydrant seeped under the foundations of the pedestals of one bent and caused a settlement of about 5 in. No cracking of the concrete girders occurred and

FIG. 2. SETTLED BENT, COLFAX-LARIMER VIADUCT, DENVER, COLO.

while the settlement was not serious it was decided to bring the bent back into place. The accompanying views show how this was done.

The posts were lifted off the pedestal blocks and a layer of grout placed under them. The posts had been bonded to the pedestals by four twisted bars, one at each corner, so that it was first necessary to cut away the concrete and expose these bars, as is being done in Fig. 1. (In the later constructed bents these dowels were omitted.) On the concrete footing a grillage of 12x12-in. timbers was built, held together by steel rods as shown in the detail in Fig. 3. To avoid the possibility of splitting off the 12-in. ledge on the concrete footing the rods were taken up so tightly that the friction between the concrete posts and the timbers sustained a large part of the load.

On top of the 12x12's a 12x16 was placed, which carries the jacks. The jacks bear against two 12x12's which carry four 12x12 posts held in place by four yokes of 12x12's and ¾-in. rods, as shown in Fig. 3. The load is transferred to the structure by two struts of 12x16 hard wood resting against the haunches of the

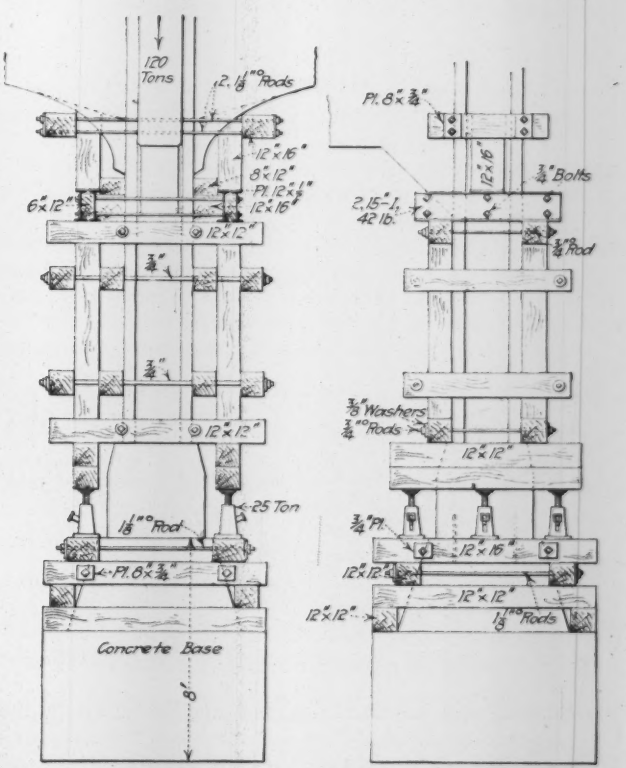


FIG. 3. DETAIL OF GRILLAGE, INCLUDING TWO 70-TON AND FOUR 25-TON JACKS

arched-end girders. These struts are prevented from slipping by four 2⅛-in. rods.

When the accompanying views were taken only one post had been lifted from the pedestal. Six jacks totaling 240 tons were required; two 70-ton jacks in the middle, as shown in Fig. 3, and four 25-ton jacks on the ends. The post finally separated from the pedestal with a very perceptible snap. The work of raising the other posts was suspended until enough jacks could be secured to lift all four posts simultaneously.

High Steel Prices Increase Use of Concrete Bridges

The Minnesota State Highway Department has found that with present high prices of steel reinforced-concrete bridges can compete quite successfully with steel bridges, especially where short spans are required. In a recent issue of the *Minnesota Highway Journal* the following analysis of the situation is given:

During the present year two plans are being furnished for nearly all bridges under 30 ft. in length. In each case one plan is for the beam span on reinforced-concrete substructure, while the other plan is for the reinforced-concrete bridge throughout, the substructure in both plans being practically alike.

The results of the bids so far indicate that 60% of these bridges will be built according to beam-span plans, while 40% will be reinforced concrete throughout. In comparison with last year the reinforced-concrete bridges have made a big gain over the beam span.

In one case early in the year, when the office was not crowded with work, three different plans were prepared for a bridge 250 ft. long. The location of the bridge was such that any one of the three schemes submitted could be built for about the same cost. It was thought that by having three different plans on file more bidders would be attracted. The plans prepared were as follows: (1) A series of six reinforced-concrete jack-girder spans on reinforced-concrete piers. (2) Two 75-ft. truss spans and two beam spans on reinforced-concrete piers. (3) Eight beam spans on reinforced-concrete piers.

On the three schemes ten contractors submitted 19 bids, and the lowest responsible bid was on scheme 1, which was accepted. It was quite surprising to learn that the accepted bid was below the estimated cost, and that the reinforced-concrete design won out over the two steel designs, one of which it was generally conceded would be the winner.

New Standard Irrigation Weir

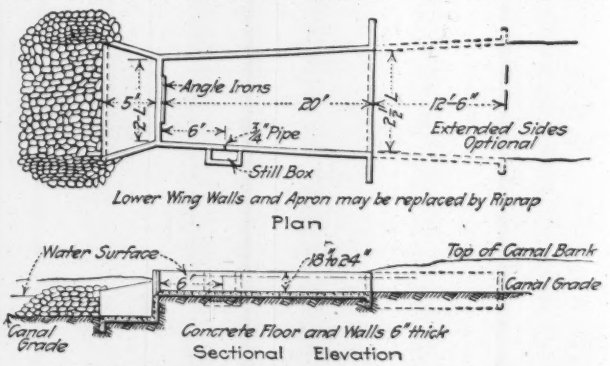
Experiments have been conducted by the United States Department of Agriculture in the Fort Collins hydraulic laboratory (run in coöperation with the Colorado Agricultural Experiment Station) to develop a standard weir that would be self-cleaning, require a minimum amount of labor and material for construction, measure discharges with an accuracy commensurate with irrigation-service needs, and be easily operated by an ordinary person.

Such a weir has been designed and is shown in the accompanying sketch. The following specifications covering it are reprinted from the *Journal of Agricultural Research*, Mar. 13, 1916 (an article by V. M. Cone, Irrigation Engineer, Office of Public Roads and Rural Engineering).

The weir notch is rectangular in form, with sharp crest and sides. The floor of the weir box must be level with the crest, and it is therefore convenient to use an angle iron for the crest, embedding one face of the angle until flush with the surface of the floor, the other face of the angle extending downward. The sides of the weir notch may also be made of angle iron placed in a vertical position, with one end extending below the crest and one face of the angle against the angle-iron crest. The angle can then be attached to the weir bulkhead through holes placed in the other face. This arrangement is durable and inexpensive and will meet the requirement of sharp crest and full lateral expansion for the escaping stream of water. The grade of the canal downstream from the weir must be low enough to give free fall and complete aëration to the nappe.

The floor of the weir box must be level throughout, and there must be no sudden or decided differences in elevation between the floor and the grade of the channel of approach. The weir box must be placed in the center of the ditch, so the axial line of the box corresponds with the axial line of the canal, in order that the water may enter the weir box in straight lines. The width of the weir box must be twice the length of the weir crest (2 L) at the plane of the weir, and two and a half times the length of the weir crest (2½ L) at a distance of 20 ft. upstream from the plane of the weir. The standard tests were made with a weir box 32.5 ft. long, except for the 4-ft. weir, and the sides were extended at the angle indicated. However, for the 1-ft. weir at least, the discharge through a box 32.5 ft. long with sides set to the standard dimensions is within 1% of the discharge obtained by placing 90° wings at the end of a similar box 20 ft. long. The use of 45° wings will cause an error of about 2½%. Therefore the weir box for the new irrigation weir should be made with sides spaced 2 L at the plane of the weir and 2½ L at 20 ft. upstream from the weir, with the sides continuing at this angle until they meet the banks of the ditch or canal; or the box should be only 20 ft. long with wings to connect the sides of the box with the canal banks, and these wings should form an angle of 90° with the axis of the weir box. The 90° wings give a discharge about 1% greater than with the extended sides for a head of 0.2 ft. and about 1% less for a head of 1 ft.

Extending the sides of the weir box until they are the full size of the canal will give more accurate results, but this accuracy may not be required, and the saving in cost of construction due to the shorter length of the weir box with



NEW STANDARD IRRIGATION WEIR

wings may be more desirable than the 1% of accuracy in measuring the water. Unless the canal bottom is easily eroded or scoured, it would not be necessary to extend the floor of the weir box beyond 20 ft., even if the sides of the box are extended.

The comparatively high velocity of the water flowing through the weir box causes a wave action and generally disturbed condition of the water surface, which makes it quite impossible to determine the head h in the open weir box. Any stilling device placed in the weir would interfere with the action of the weir, and it is therefore necessary that a still box be placed outside the weir box and connected through the side of the weir box with one or more 1-in. pipes located 6 ft. from the plane of the weir. The pipe should be placed near the floor of the weir box to insure its being submerged for low heads, and care must be used to place the pipe normal to the side of the weir box, and not normal to the axis of the box. If the pipe is pointed downstream the velocity of the water in the weir box will cause a suction action which will make the water surface in the still box lower than that in the weir box. If the pipe is pointed upstream, there will be a velocity head added to the actual water level in the weir box, and the water in the still box will be higher than that in the weir box. Although no sand or silt will accumulate in the weir box, regardless of the amount carried by the stream, silt may be deposited in the still box and clog the connection pipe unless it is cleaned regularly. By making a deep still box, space will be provided for such silt accumulation and therefore less frequent cleaning will be required. The still box should have inside dimensions of at least 1x1½ or 2 ft., with such depth as is necessary. The head in the still box may be determined by means of a scale, a hook gage, or an automatic registering gage.

The new irrigation weir may be constructed of lumber, but the design is such that it may be easily constructed of concrete. There would be no difficult form work required for the

concrete, and it would make an inexpensive, durable and satisfactory measuring device, especially if the angle-iron sides and crest of notch were used in connection with the concrete box.

The formula for discharge past this weir is:

$$Q = (3.83 - 0.07L)\ Lh$$

Discharge tables for weirs of 1 to 4 ft. length and for heads of 0.2 to 1.25 ft., by ⅛ in. steps, have been computed and published (in a bulletin reprint of the article already noted). In addition to the requisites already mentioned as kept in mind during the development of this weir, the following advantages are reported:

No lowering of canal grade or building-up of banks is required in construction. The device is not patented. It cannot easily be tampered with or injured so as to alter its discharge.

Stump Pullers Operated by Hand and Horse Power

The hand-power stump puller shown in Fig. 1 is of the clutch type, and is manufactured by the A. J. Kirstin Co., of Escanaba, Mich. The pulling cable passes through a pair of clutch boxes operated by the movement of a long lever. As the lever is swung to and fro the clutches are gripped, moved and released alternately, so that the cable is held firmly by one clutch while the other is released and moved back for another grip and pull. The operation of the pulling setup shown is as follows:

The puller *A* has one end attached to the cable *B* by means of a trip hook *C*. The cable is passed around an anchor stump, then to the block *D* and returns to pass through the clutches on the puller. A second cable *E*, has a ring socket at one end which is hooked to the block *D*; it passes through a second block *F* and is then led to the anchor stump, to which it is secured by looping under a hook *G* on the end of the cable. A third cable *H* with a hook at one end and ring socket at the other is looped around the stump which is to be removed and secured by its hook, in the same way as at *G*. Its free end is passed around a take-up or cable-shortening device *J*, which is hooked to the second block *F*, and provides for taking up all slack between the block and the stump.

For heavier pulls, a third cable and block, similar to *EF*, may be used, with the take-up hooked to this block, as shown in the top diagram of Fig. 2. For comparatively light pulls, only the first block and the two cables *B* and *H* are used, the latter being connected to the block *D* either directly or through the take-up.

Other hand-operated stump pullers are of the drum type. One of these is the "K" device manufactured by Walter J. Fitzpatrick, 182 Fifth St., San Francisco, Calif. In this the cable is wound upon a small drum mounted in a frame and driven by pawl and ratchet gear by means of a long lever handle. From the drum the

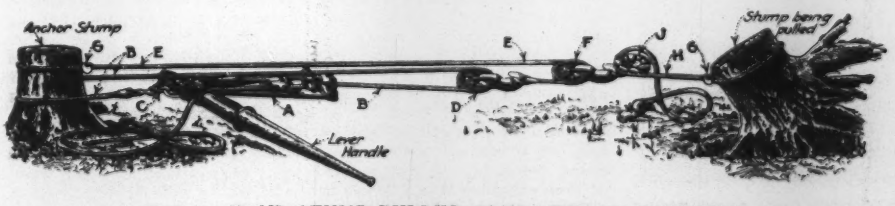


FIG. 1. HAND STUMP PULLER OPERATED BY CLUTCHES

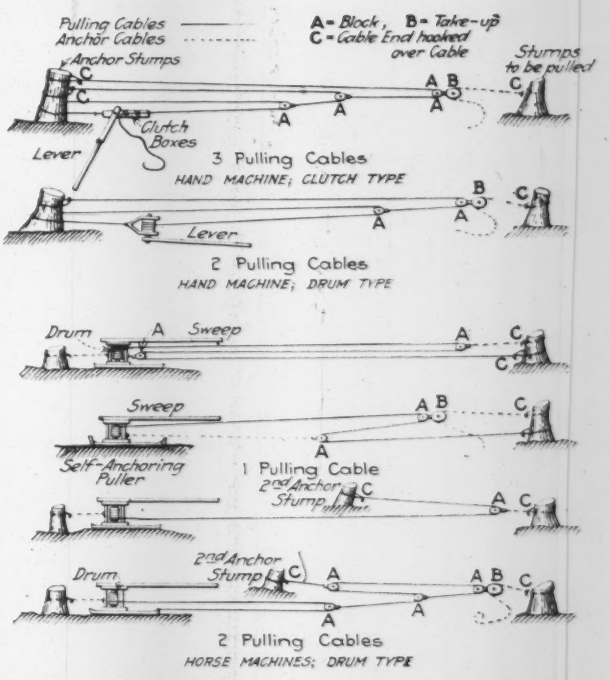


FIG. 2. ARRANGEMENT OF OUTFITS AND RIGGING FOR PULLING STUMPS

cable is led through a block and then returns, passing around the anchor stump and having its end attached to the frame carrying the drum. This arrangement is shown in the second diagram in Fig. 2.

Horse-operated stump pullers are drum machines, having the drum mounted upon skids or a truck and driven by means of one or two sweeps to which horses or teams are hitched. The frame carrying the drum may

FIG. 3. LETTING HORSES DO THE WORK

be hitched to an anchor stump, or secured by embedding its sills in the ground and driving heavy stakes at the ends. Fig. 3 shows an outfit by the Faultless Stump Puller Co., of Cresco, Iowa. Similar machines are made by numerous makers, each having its own special features.

The arrangement of the cable rigging is adapted to the power required, and some typical arrangements are shown in Fig. 2. Power is multiplied by attaching additional lines and blocks as indicated. When not required for this purpose, these extra parts may be used as snatchblocks to guide the main cable clear of obstructions

or to keep it down near the ground so that the horses can step over it.

Various special devices are used in stump-pulling work. When stumps are small or too short and rotten to hold the cable loop or snare, a steel stump hook or root plow is used. This is of V-shape, with one leg horizontal and hitched to the cable; the other leg is inclined and ends either in a heavy point or in a broad-forked end to engage the stump or root. Handles on the back enable a man to guide the plow into position as it engages the stump. There are also root hooks (like ice tongs) for pulling short and rotten stumps.

For clearing brush, there is a device having three or four short ropes or chains attached to an iron head which is hooked to the pulling cable. These ropes are looped around the stumps or trunks, and enable a large amount of the brush to be removed at each pull. Where the stumps are to be piled for burning, a "skidder" or light portable derrick on skids is used.

NOTES

Traffic Signal Lamp Posts have been made to answer the purpose of traffic policemen in St. Louis and some other cities. In St. Louis traffic lights have been placed in the middle of several of the busiest streets and boulevards for the purpose of separating the two streams of traffic, indicating a street-car crossing and providing an "island of safety" for pedestrians. The light shown in the accompanying view at 12th and Locust St. is typical of the style of light generally used.

TYPICAL TRAFFIC SIGNAL LAMP POST, ST. LOUIS, MO.

At this particular intersection there are two lights, but the one shown is placed just a short distance south of Locust St. about one-third width of the roadway from the east curb. This is for the purpose of separating northbound vehicular traffic into two parts, that going to turn west on Locust St. passing to the west of the light, and that going to continue north on 12th St. or turning east on Locust St. passing to the east of the light. An explanatory sign indicating this is mounted on the lamp post. Just north of Locust St. and about one-third width of roadway from west curb is located the other light to separate the southbound traffic in a similar manner. Generally the traffic red lights do not require any signs.—William Holden, Engineer, Department of Streets and Sewers, St. Louis, Mo.

Cleaning Up a Wide Ditch with a trenching machine is shown in the accompanying view of a Minneapolis water-works pipe-laying gang. The trenching machine cuts the initial trench, which is widened by five men with long bars, working alongside the machine. Three men with shovels in

TRENCHING MACHINE RAISING LOOSE MATERIAL

the bottom of the trench shovel the loose material into the buckets and leave the trench leveled to grade for pipe laying. The trenching machine is owned by the municipal water-works department. The work shown is in 20th Ave., Minneapolis, in connection with the installation of a new 46-in. lock-joint steel main.

Blasting in Clay—Blasting down the face of clay pits which have been worked by pick and shovel is reported by L. C. Longstreet, a North Carolina operator. A line of holes, 3 ft. deep, was put down 6 ft. back from the face and each loaded with 1½ sticks of 40% dynamite. In some brickyard pits a large amount is blasted out in the fall and allowed to freeze and thaw through the winter.

Steel Steps for Manholes and Catchbasins at Pueblo, Colo., have been reduced in cost from 35 to 20c. by changing the stock and the method of manufacture. Formerly the steps were forged from ½-in. rods. They are now made from ⅜x1-in. flat bars, cut, bent and twisted cold. The steps measure 9 in. wide by 4 in. broad, and the ends for insertion in the brick are 9 in. long, not including a turned-up portion 1 in. high. D. P. Gaymon is city engineer of Pueblo.

To Sterilize Camp or Household Water—Following is a satisfactory method of sterilizing small quantities of water for use in camps and summer cottages, that is recommended by public-health authorities: Take a level teaspoonful of chloride of lime and rub it up in a teacup with a little water to a thin paste free from lumps. This should then be made up to a cupful and diluted with three more cupfuls of water and poured into a bottle and tightly corked. This is a stock solution and will keep for a week if tightly corked. A teaspoonful of this stock solution should be added to each 2-gal. pail of drinking water, thoroughly mixed, and allowed to stand for a few minutes. This will give about one-half part of free chlorine to a million parts of water, which will destroy all typhoid- and dysentery-producing germs in 10 min.

Editorials

New York Leads United States in City Zoning Progress

In breadth of scope, recognition of fundamental principles, and attention to detail, the zoning or districting regulations governing the height and use of buildings and the percentage of lot area built upon, adopted on July 25 by New York, places that city far in the lead of the cities of the United States, if not of the world. Boston and some other American cities, including New York itself, have limited the height of buildings over either a small part of their area or as regards some classes of buildings; various American cities, again including New York, have prohibited covering more than stipulated percentages of lots with certain classes of buildings; Los Angeles divided its area into residence and industrial districts in 1909 and has been followed in this by a number of other American cities; in Germany and some other European countries all these matters have been regulated for decades and some of them for a century or more; but of American cities, New York has now placed itself far in the lead in all the particulars named.

The adoption of these restrictions by a 15 to 1 vote of the Board of Estimate and Apportionment on July 25 was the culmination of two years' work by a Commission on Building Districts and Restrictions, authorized by a legislative amendment to the city charter in 1914, preceded by a year of study by a Heights of Building Committee, aided by the experts of the City Plan Committee of the Board of Estimate. Before the restrictions were put in final form a preliminary report was published and public hearings by geographical districts were held. Practically no opposition to the general principles were voiced and comparatively few to the details of the plan.

Broadly, the scheme establishes *Use, Height* and *Area* districts. *Use* districts are divided into Residence, Business, Unrestricted and Undetermined (see *Engineering News*, April 13, 1916, p. 700). *Height* districts are of five classes, with building heights limited to 1, 1¼, 1½, 2 and 2½ times the width of the streets, measured at the building line, but with possible greater heights back of that line by the use of setbacks or by mansards. *Area* districts are of four classes, permitting built-over areas at the first story ranging from 100% in business to 50% in parts of residence districts having detached or semi-detached houses.

Obviously, the area and height limitations are in the interest of light, air, and health. The use limitations serve the same purposes in varying degree. All the restrictions will play a vital part in the harmonious development of the city plan and will contribute to the stability of real estate values, including the prevention of blighted districts where property values fall because of unregulated development. (See editorial note, "Merchants Turn City Planners," *Engineering News*, March 16, 1916, p. 523.)

Judiciously administered, this districting plan substitutes public control of city growth under a systematic plan designed to promote the interests of the whole city for haphazard, piecemeal private control aimed solely to benefit individual property owners, often to the ultimate damage of other owners of private property.

The change is bound to be salutary. Under the leadership of New York and with stimulating examples in other parts of the United States, as well as in Canada, the zoning movement and more intelligent city planning generally is bound to make great progress in America within the next decade.

Dr. Travis and the Septic Tank

The intermediate step between the one-story septic tank as Cameron left it and the two-story septic tank of Imhoff is little known and soon forgotten in this country. The interesting recent example of a hydrolytic or Travis two-story septic tank at Luton, England, described by Borough Surveyor Tomlinson in the first pages of this issue, affords a fitting opportunity to call attention to the notable part played by Dr. W. Owen Travis in the evolution of the septic tank.

The main and probably the most useful conception of Travis, or rather his working out of the principle first tested under H. W. Clark at the Lawrence Experiment Station of the Massachusetts State Board of Health, was to separate the settling and the digesting or liquefying functions of the septic tank. Dr. Travis carried on sedimentation in an upper and digestion in a lower compartment, to prevent gas rising and carrying sludge up into the settling sewage. His lower compartment, however, extended upward in a central longitudinal space between the two settling chambers, and, at the original working installation at Hampton, 12½% of the total tank influent was carried through the settling compartment, which had cross walls provided with ports to lessen the carrying forward of gas-buoyed sludge.

The second and apparently more original if less valuable feature of the Travis tank was embodied in his colloidors, or multiple surfaces for depositing sludge and adherent matter in addition to the deposits on the sides and sloping bottoms of his settling compartments. The hint for this also came from Lawrence.

When Dr. Imhoff took the Travis tank in hand for the Emscher District in Germany he adopted the two-story idea, but made an absolute separation of the settling and digesting function by carrying his influent through the upper compartment only. Dr. Imhoff did not adopt the colloidors. He made various other changes in design and operation. His tanks in the Emscher District impressed Rudolph Hering and other American engineers as superior to those of Dr. Travis. Thus it was that the Imhoff rather than the Travis tank was brought to America and its use has since spread through the country, much as did the Cameron tank some years earlier.

Notwithstanding the vogue of the Imhoff tank in Germany, the United States, Canada, and elsewhere, it ap-

pears to have got no footing in England for some years and very little at all. Nor has the Travis tank been as widely used in England as would naturally be expected—at least the one built some years ago at Norwich and the initial working-scale tank at Hampton-on-Thames have been the only examples commonly mentioned here.

In view of the foregoing facts and opinions it seems probable that Mr. Tomlinson's article on the Luton hydrolytic tank will be welcomed by sewage engineers generally, both in and of itself and because it brings to the front the name of a man who does not always get credit alongside of Cameron and Imhoff for playing an important part in the evolution of the septic tank.

Wanted: A State Road Policy

The State of New Jersey has been building good roads through state aid longer than any other state in the Union. The state and its counties, cities and towns have all been spending money liberally for a quarter of a century past on good road construction and maintenance. Yet the condition of New Jersey roads is now so bad that the recent legislature voted to borrow $7,000,000 on state bonds for road reconstruction, provided this bond issue receives approval by popular vote at the November election. In addition to this, the legislature created a commission with broad powers to investigate the entire road question and formulate if possible a better policy for the state to follow in highway construction and maintenance.

This commission has begun its work by sending out a circular letter of inquiry, which was reprinted on page 187 of *Engineering News* of July 27. It outlines in brief terms the difficult features of the road problem that faces not only New Jersey, but many other states.

In all the good-roads literature that has been poured out in such profusion for more than a score of years the chief appeal has been for more money to build good roads. Volumes of figures have been published to prove that the community which spends money liberally to build good roads has done all that is necessary to place it on the highway to wealth. New Jersey and its communities have done all this. The state expended last year for road construction and maintenance $1,263,000.

This money has not been expended ignorantly. New Jersey roadwork has had as good engineering talent engaged on it, probably, as any state in the Union. A most fortunate feature of New Jersey's past road policy is that the state's expenditure on roads has been money raised by direct taxation and not money borrowed on bond issues. While New Jersey counties and towns have a considerable amount of road bonds outstanding, the state as a whole is not in the difficult position of some of its sister states which have borrowed money on long-term bond issues to spend on roads that are already worn out.

On the other hand New Jersey is unfortunate in this respect: Because of her early activity in road construction she has expended many millions of dollars in the construction of water-bound macadam roads, a type of construction admirably suited to the horse-drawn traffic which existed at the time the roads were built, but wholly unsuited to the motor traffic of the present day. The various makeshifts that have been tried during the past decade to enable a water-bound macadam road to stand motor traffic have involved large expense and have generally proved unsatisfactory in service. Fortunately, the last few years have seen the development of types of road surface which can withstand the stress of motor traffic and whose first cost and cost of maintenance are not prohibitive.

The first item in a road policy for New Jersey, therefore, should be the adoption of permanent types of roads in place of the types formerly used. In order to do this, however, and in order to insure the efficient and economical conduct of road construction and repair, New Jersey needs to reform its system for the conduct of its highway work. This reform need not be a very radical one, for New Jersey has had for years a state highway department. What is needed is to increase the authority and responsibility of this highway department so that it may have power to supervise the roadwork carried on by counties and by towns, as well as the work carried on by the state. In order to do this the department will need to be organized on a permanent basis, with a legal tenure of office for the engineers employed in the department and with rigid exclusion of politics from its organization. This reform may sound like a simple thing to effect, but it is far from simple in the majority of states, and it remains to be seen whether it will prove simple in New Jersey.

A Revolutionary Improvement in Concrete Road Building

For a number of years engineers interested in road construction made pilgrimages from all parts of the country to Wayne County, Michigan, to see the famous concrete roads there and to learn how it was possible to use ordinary concrete as a paving material for country roads. An article printed in the last issue of *Engineering News*, describing the remarkable results attained in building concrete roads and streets at Sioux City, Iowa, will doubtless be the means of setting in motion a steady stream of engineers from all parts of the country to visit Sioux City and see for themselves how these results are obtained.

It is no exaggeration to say that the concrete roads and streets of Sioux City represent an important advance over anything heretofore attained in concrete-road construction within our knowledge. The main reason for this improvement in the character of the concrete is found without doubt in the method adopted for finishing the roads. This method was described by the City Engineer of Sioux City, T. H. Johnson, in *Engineering News* a year ago and was more fully explained in last week's issue. Briefly stated, it consists in applying to the finishing of a concrete road the method long in use for giving a hard, wear-resisting surface to concrete sidewalks. A dry mix of cement and sand is applied to the surface of the concrete after it is spread on the subgrade and smoothed off. The quantity of this dry mix is small, just sufficient to absorb the surplus water; and this mixture is thoroughly incorporated with the upper layers of the concrete by rubbing with heavy floats.

Concrete experts all agree that to make the strongest concrete it should be mixed with the smallest amount of water to permit thorough mixing and that surplus water left at the surface of the concrete when it hardens forms laitance and leaves a weak, friable skin on the surface

when the concrete sets. Under the present standard method of finishing concrete roads, however, in use everywhere, save at Sioux City, it is impossible to have the concrete as dry as it should be when the final set takes place. Enough water must be put in the mixture to permit the materials to be thoroughly incorporated; and when this mixed concrete is placed on the road, formed into shape by the strike board and floated, more or less surplus water invariably comes to the surface. The concrete is so soft that in order to finish it the workmen engaged in finishing have to lie flat on a portable bridge moved along over the surface of the road. The most they can do is to give the surface a light smoothing.

In contrast with this is the method used by Mr. Johnson at Sioux City, where by applying the dry mix of cement and sand to the surface just as soon as the road is brought to shape, the surplus water is drawn out by capillary attraction and the concrete is at once so stiffened that the workmen engaged in floating can walk upon it and make hardly an impression. The concrete then receives a thorough compacting and kneading from the heavy wooden floats, such as it is not at all possible to give to a road finished from bridges in the usual manner.

It is not at all strange that the surface of a concrete road finished in this manner should be far harder and more durable than that of a road finished in the ordinary way. Further than this, the remarkable freedom from cracks of the Sioux City roads indicates that the beneficial action of the heavy float is felt through the whole thickness of the concrete layer.

It is worth while remarking in this connection that the quality of the sand used in the concrete and in the finishing may very likely be another factor in the remarkable resistance to wear of the Sioux City roads. The need of greater attention to the sand for concrete ought to be pretty well known by now to engineers everywhere; but it seems important to emphasize that for concrete-road construction and especially for the wear-resisting surface of a concrete road it is essential that a *silica* sand be specified. In certain sections of the country the sand in general use for mortar and concrete is composed chiefly of particles of limestone instead of silica. Concrete made with such sand cannot be expected to show a great resistance to abrasion and is sure to be much more slippery than concrete made with a sharp silica sand.

Of course we do not attempt to say that the methods in use at Sioux City are above criticism or incapable of further improvement. Many engineers for example will probably think that the use of the strike board to give the pavement its cross-section is a better plan than relying wholly on the skill of the workman. Many will question also whether so much expense in floating the finished pavement is warranted. That the Sioux City system of construction, as a whole, however, has proved itself thoroughly satisfactory, is proved by the fact that four successive incumbents of the City Engineer's office have continued its use.

Finally, it is well worth pointing out that the high quality of concrete in the Sioux City pavements is being obtained at a trifling expense. In fact, the contract price per square yard prevailing there is little, if any, higher than that in many other cities where concrete roads are being built. There are many cases where improved methods and materials are advocated for engineering work, and the engineer-in-charge may legitimately reject them on the ground that the increased expense involved is not warranted by the improvement in results. To build concrete roads by the Sioux City plan, however, costs so little more than it does to build them in the ordinary way and the results obtained are so vastly better that no reason appears why the Sioux City method should not shortly become standard practice.

Street Cleaning in New York

According to the recent report of Richard T. Fox on an examination of the New York Department of Street Cleaning, the status of the street sweeper has much improved in the last few years. The laborer who wields a broom in New York City today receives an annual salary of $840, has two weeks' annual vacation, sick leave and a life pension if he reaches a fixed age limit after 20 years' service. There are not a few college graduates who can do no better.

Mr. Fox truly says that such favorable working conditions "have no parallel in the history of the business world for laborers of this class." Yet instead of the body of alert, active, interested workers one would expect to find, the investigator was surprised to find a large number of old men, and aside from these many more who were physically below the standard such excellent rewards should command. A general inefficiency and want of discipline were evident, with a corresponding lack of thoroughness of the work.

For street-cleaning service the three more important boroughs of New York City—Manhattan, Brooklyn and The Bronx—pay the sum of $8,000,000 yearly, exclusive of snow removal. The number of permanent employees is approximately 7,000. Snow removal costs add from $2,000,000 to $3,000,000 more a year. The administration of this work obviously requires an expert, and consequently Mr. Fox's first recommendation is that the offices of Commissioner and Deputy Commissioner be freed from politics.

Other recommendations for the administration of the department are that the Commissioner be given greater latitude in making expenditures and that there be greater coöperation between this and the various other city departments, particularly on the part of the courts and police department in enforcing the ordinances against littering the streets with paper and other refuse. The equipment of the department and the cost-keeping system are held to be inadequate and not up to date.

The foregoing recommendations apply to the department as a whole, but some of those which apply only to the installation of a model district are of almost equal interest. Among these are that hand sweeping only be employed; supplemented by flushing or washing with water under pressure; that the sweepers wear leggings; that they polish their shoes daily; that sweeping the sidewalk be included in their work and that permanent stations or boxes be provided at convenient intervals for storing the sweepings between collections.

With polished shoes, leggings and a brown khaki uniform the street sweeper's dignity ought to be raised at least to that of the average citizen; and with the strong arm of the law to protect him from the unnecessary labor entailed by careless people who throw refuse in the street, the work ought to be attractive; the commissioner should be given sufficient latitude to make it efficient.

Letters to the Editor

Supporting the Mississippi Levee System

Sir—The editorial in your issue of June 29, "The Mississippi Levee System Sustained by the Courts," shows a clear conception of the problem and a correct understanding of public sentiment in this section. The suit that you refer to was never taken very seriously by the levee district officials. It was brought by owners of lands along the narrow margin between the levees and the river banks. These lands have always been overflowed at high water (except small areas where hillside slopes approach the river), and were purchased by their present owners with a full knowledge of this fact. It is true that confinement of the flood water between levees causes deeper inundation of these lands than formerly; but they are worth more now than they were before levees were built. This increased value results from their proximity to the highly developed country immediately behind the levees, which only recently was largely a wilderness; and from the fact that when these places are submerged, their owners can now drive their livestock over the levee to safety, where formerly they built mounds or rafts for them, or boated them to another state.

You rightly say, "This minority finds representation in certain Southern newspapers." It would be well for this statement to be weighed in connection with several newspaper quotations appended to one of the Congressional speeches referred to below.

Recently the writer received a circular letter from Congressman James A. Frear, inclosing various extracts from the *Congressional Record,* and the Chicago *Tribune* article from which you quote, all in opposition to the Humphreys flood control bill. The same letter and inclosures will doubtless reach many other engineers and it is gratifying that your timely editorial will give them another view of the matter. It is evident that the two Congressmen and the Chicago *Tribune* have a misconception of the whole situation on the lower Mississippi River.

For the benefit of engineers who have not been in touch with the organization and work of the Mississippi River Commission, it should be stated that appointments to membership in that body are made by the President of the United States, and membership continues until death, resignation or removal for cause. No membership has yet been terminated except by the first two methods. In the personnel of this Commission are several United States Army engineers, several civil engineers and one lawyer. In the entire personnel of this Commission since its organization few of its members have been residents or property owners on the lower Mississippi.

The Mississippi River Commission has made surveys, soundings, discharge observations and studies for more than thirty years, and has pronounced that the only practicable way to control the floods on the lower Mississippi is to revet the caving banks and complete the levee system. For thirty years its members have been as competent and have had as good opportunities as any one possibly could have to study and compare various plans of flood protection and control, and it has been one of their duties to decide upon the best plan for the Mississippi River.

The completion of the levee system is not a land reclamation scheme. The reclamation of the lands on the lower Mississippi has already been effected. These lands are now protected, and have been for years, by almost continuous levee lines from Cairo to the Gulf, and behind these levees are large sections of highly developed country, with frequent towns and busy railroads. The existing levees were paid for in large part by the landowners. The United States Government has, for about thirty years, made some contribution toward this work; but it is believed by the writer that, up to this time, the total contributions by the Government do not much exceed the total cost of replacing levees that have caved into the river because the banks have not been revetted to prevent the channel from shifting. The lands protected by these levees are so far from being unreclaimed, that fully 90% of the area behind the levees was free from water during the flood of 1916, which made record stages on a large part of the river. Only a small percentage of these lands is owned by syndicates.

Occasionally the writer has thought proper to enter discussions of the levee question in civic clubs, in the hope of preventing the adoption of resolutions proposed by the "vociferous minority." It is remarkable that a man in almost any other walk of life can expect a hearing on an engineering subject and be accredited with disinterestedness; but that an engineer's arguments meet with the response, "of course you favor levees, because you are an engineer." The idea is prevalent in this section that in some way all engineers will profit directly if Congress makes large levee appropriations.

With an apology for mentioning personal affairs, but realizing that upon every engineer favoring Government appropriations for levees rests the burden of proof that he is disinterested, the writer will state here that he owns no property that would be benefited by levee construction, from Government appropriations, that he is not employed on levee work and never expects to be employed on that work again; that he has not been for fourteen years; and that if he were or expected to become a levee district engineer, he would be working against his own interest to favor levee building by the Government, because their expenditures are supervised by their own engineers. (Twenty years ago the writer left the Government service, having then about reached the limit of salaries for civil engineer assistants.) For the Government to finish the levee system will put a number of levee district engineers "out of a job."

The writer is convinced that no expenditure of like amount which this nation could make, except for military and industrial preparedness, will give larger returns in an increase of the nation's wealth and prosperity, than

that necessary to make this valuable section of country absolutely secure against Mississippi flood water. That levees and revetment will accomplish this has been conclusively demonstrated. A. L. DABNEY.

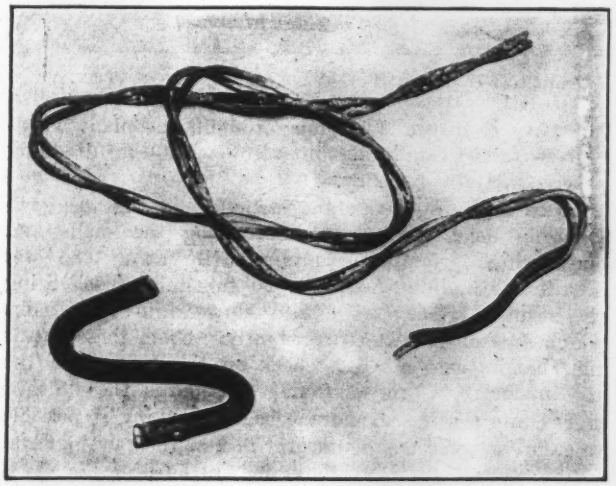
Porter Building, Memphis, Tenn., July 3, 1916.

Gypsum and Its Effect on Embedded Steel

Sir—There seems to be a somewhat popular supposition that gypsum in contact with metal will cause progressive corrosion of the latter. Certain facts available show the contrary to be the case, and in justice to the material I think they should be put on record.

The most recently revised building codes, as well as the 1915 edition of the National Board of Fire Underwriters recommended standard building regulations, include the use of gypsum for the fire-protection of steel supporting members. The advisability of admitting gypsum for this purpose was the outcome of investigations conducted during 1911 by the "Joint Committee on City Departments" of New York under the direction of Chairman Ira H. Woolson, Consulting Engineer for the National Board of Fire Underwriters.

A subcommittee conducting a special investigation of the question of possible evil effect on the embedded metal caused the uncovering of the reinforcing steel used in the construction of the Metropolitan floor (gypsum floor) which formed the roof garden of Hammerstein's Victoria Theater in New York City. The part of the construction cut into was under the stage and near a tank of water which was used for diving and other purposes. This location it was thought would disclose the worst possible conditions, as corrosion of the embedded steel (if any) would be aggravated by any possible and continued moisture.



STEEL WIRE AND HOOKS NOT CORRODED AFTER EMBEDMENT IN GYPSUM FOR 15 YR.

S. G. Webb, Secretary of the Gypsum Industries Association, was present during this investigation and states that the reinforcing consisted of S-hooks secured to the I-beam flanges, to which were attached the twisted wires forming the reinforcement. The S-hooks were not galvanized or covered in any manner with a protective coating of any kind, though the twisted wires were galvanized. The steel-supporting I-beams, the S-hooks and the twisted wires were buried in the gypsum construction for a period of about 15 years and when taken out were all in excellent condition. The S-hook and twisted wires, shown in the accompanying view, were taken from the construction referred to.

During August, 1912, a fire test was made upon a Metropolitan floor (gypsum) at the testing station of Columbia University, for the City of Toronto, Can. This construction (a 4-in. thick poured reinforced-gypsum slab) successfully passed the 4-hr. load and fire test required by the New York Building Department. After this test the entire construction was torn out, and the wreckage thrown into a scrap yard where it remained exposed to the elements of the weather through the following winter, spring and summer.

An examination of this scrap made by Mr. Woolson, Prof. James McGregor, of Columbia University, H. B. McMaster, S. G. Webb and representatives from the New York Building Department, disclosed the fact that any metal which remained embedded in the gypsum mixture was not corroded, but all metal which had been exposed to the weather during the months of exposure of course showed signs of active and progressive corrosion.

The United States Bureau of Standards is conducting a test to determine the relative protective value of various coatings upon metal lath. This test was commenced upon full-size panels in 1911. A progress report by J. C. Pearson, of the Bureau, gives the following facts:

Assuming the numeral 10 as indicating perfect condition—no rust—the following was found with reference to the gypsum plasters:

Lath Galvanized Before Expansion—9.6 for gypsum plaster, 9.8 average of all plasters.

Painted "Special" Iron Lath—9.0 for gypsum plaster, 9.3 average of all plasters.

Painted Steel Lath—9.0 for gypsum plaster, 9.1 average of all plasters.

Sherardized Lath—7.5 for gypsum plaster, 8.3 average of all plasters.

Unpainted "Special" Iron Lath—7.25 for gypsum plaster, 7.0 average of all plasters.

Plain Steel Lath—7.5 for gypsum plaster, 7.6 average of all plasters.

Commenting on the showing of gypsum plasters, Mr. Pearson states:

> Examination of the table will show that the condition of the lath under the three representative brands of gypsum plasters is almost exactly a general average condition of all the interior panels; that is, the corrosive action of the straight gypsum plaster appears to be less vigorous than was anticipated.

These facts and experiences prove two things: (1) That gypsum is a fixed salt, and neutral in action; in other words, the SO_3 is completely combined with CaO in the anhydrous form of gypsum of $CaSO_4$. Dr. W. F. Hillebrand, United States Bureau of Standards, says: "It is incorrect to speak of a sulphuric acid content of gypsum. The properties of sulphuric acid are lost when the acid is fully neutralized as in gypsum." (2) That when gypsum is used upon metal lath, or is in contact with structural metal for the purpose of fire protection, and when air and moisture are prevented from coming in contact with such metal, no progressive corrosion can occur.

VIRGIL G. MARANI,

205 W. Monroe St., Chicago, Ill., July 13, 1916.

Portland Cement Made from Oyster Shells

By H. Struckmann*

The extensive shell deposits found all along the coast of the Gulf of Mexico have for many years been the object of considerable research work by portland cement manufacturers, as the shells properly prepared represent the purest lime constituent possible for cement making. In the past the difficulty has been to find a process by which the shells could be pulverized preparatory to mixing them with the alumina and silica constituents as contained in certain clay deposits, but this problem has at last been solved by treating the shells under the wet process as developed by F. L. Smidth & Co., of New York City.

The new plant which the Texas Portland Cement Co. has completed at Manchester, near Harrisburg, Tex., is the result of the research work conducted in the company's laboratories at Dallas as well as the recheck in the laboratories of F. L. Smidth & Co. The location of the plant was selected at Houston, Tex., in preference to other shipping points along the Gulf coast on account of the railroad facilities available at Houston and particularly on account of the recently completed ship channel which will enable the Texas company to handle export shipments by loading the cement direct from the plant into the bottoms.

The actual construction work of the plant was started in the late fall of 1915 at which time the company's engineering force was moved from the main office at Dallas to Houston, while the plans and contracts were prepared in the office of the writer who is the company's Consulting Engineer.

The plant represents an investment of approximately $750,000 and as can be seen from the general view the various structures occupy considerable space. Described briefly, it consists of the following buildings and structures.

On the ship channel is a wharf of ample capacity to receive the shell barges, bulkheads having been provided along the shore to prevent any sliding or cave-ins of the bank which might gradually fill up the channel. From the wharf a 650-ft. structural-steel crane runway supports an electrically operated monorail crane which serves the double purpose of unloading the shells from the barges and delivering them to the mill, as well as delivering the shells from the barges to a 20,000 cu.yd. storage underneath the runway, this precaution being necessary to insure steady operation in the plant whenever weather conditions interfere with the dredging process in Galveston Bay.

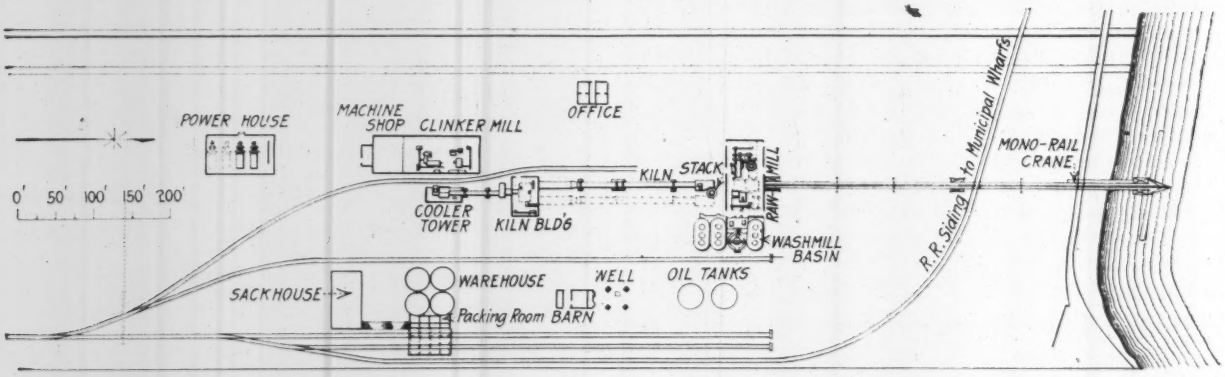


FIG. 1. PLAN OF THE NEW CEMENT MILL OF TEXAS PORTLAND CEMENT CO., NEAR HOUSTON, TEX.

Next adjoining the crane structure is the wet raw-grinding department used in treating the shells and clays which are ground preliminarily in a 0.85 wet Kominuter and finished in a 20 tube-mill, discharging the resulting finely pulverized slurry into two basins which are being agitated constantly. From these basins the chemist gets his first check on the analysis of the raw material before the pumps discharge the finished slurry into a series of storage basins, making it possible to secure a perfect and uniform mix of the raw material before delivering it to the kiln for burning.

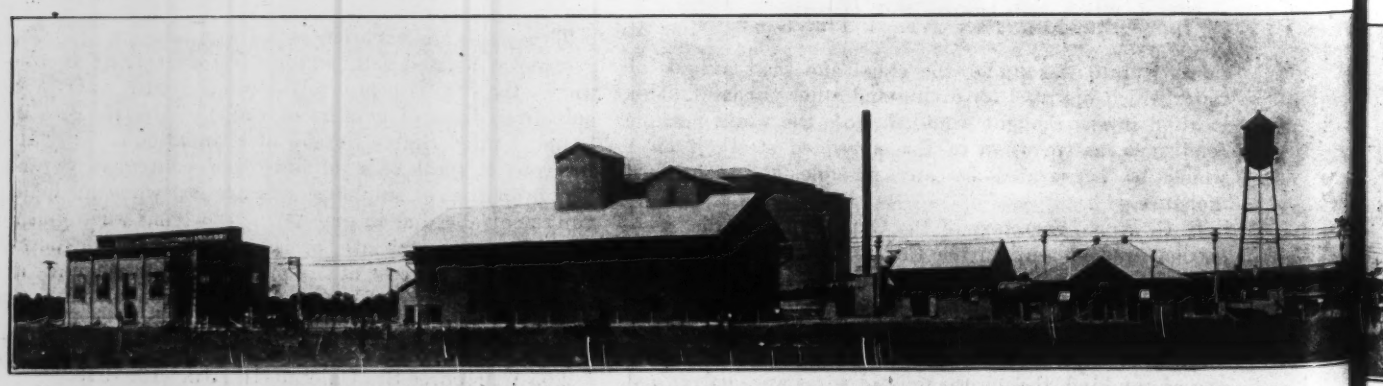


FIG. 2. PANORAMIC VIEW OF THE NEW CEMENT PLANT OF THE TEXAS PORTLAND CEMENT

*Consulting Engineer, Kansas City, Mo.

This feature in the wet process as used at the Houston plant is one of the important points which has made it practical to produce a high grade and uniform cement out of a material which in the past has had little or no value.

After the finished slurry has been analyzed, corrected and checked, it is delivered to the kiln department ready for the burning process which takes place in a 9x8x220-ft. rotary kiln operated by variable-speed motors so as to give the chemist perfect control over the burning. Residuum oil is used for fuel.

From the kiln, the clinkers are discharged into a 6x60-ft. rotary cooler where they are cooled preparatory to adding the usual amount of sulphuric anhydride which is mixed with the clinker in the form of gypsum.

The last stage in the manufacture of the cement, the clinker grinding, is handled with one No. 85 Kominuter and one No. 20 special Danula tube-mill, which represents the latest improvement in tube-mill grinding and which has shown a very remarkable saving in horsepower as compared with past known methods of tube-mill grinding.

The product discharged from the clinker mill is delivered to a belt conveyor discharging into the warehouse, which consists of four 30-ft. diameter by 60-ft. high circular concrete tanks nested together, using the ends and interspace for storage in addition to the main tanks.

The packing department, screen house and trainshed are built monolithic with the circular silos, which are elevated about 4 ft. above the outside ground level so as to permit plenty of air and ventilation in the basement containing the conveying machinery.

The power plant consists of a monolithic concrete structure large enough to accommodate four 450-kw. Diesel engine units, direct-connected to General Electric generators. Only two of the engines, which were furnished by the Fulton Iron Works, of St. Louis, have been installed for the present capacity, but all arrangements have been made to install two additional units.

In general, the plant has been equipped with the most modern and uptodate machinery on the market, and it is very gratifying that the wet process has made it possible to treat the material available and produce a high grade portland cement, incidentally eliminating the usual dust nuisance surrounding a cement plant.

While the plant has now been in successful operation for nearly two months, during which time it has turned out close to 50,000 bbl. of cement, the company has just recently started shipments under the direct supervision and recheck of the engineers of the Pittsburgh Testing Laboratory, who certify to the quality of every barrel of cement leaving the plant.

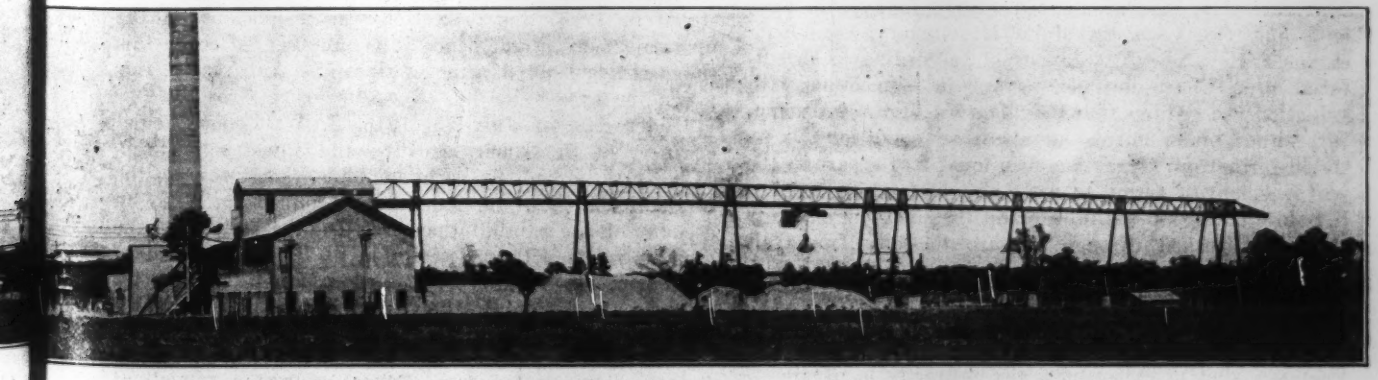
In this connection one of the important features of the plant is that by using oil for fuel and shipping kaolin in from the numerous deposits in Texas, it will be possible to produce pure white cement out of the materials used at Houston.

Two New Dams Proposed for San Diego Water-Supply

Following the destruction of the rock-fill dam at the Lower Otay reservoir of the San Diego, Calif., water-supply (noted in *Engineering News*, Feb. 3, 1916, p. 236) the city called in for consultation M. M. O'Shaughnessy, City Engineer of San Francisco, to make plans and specifications for dams to create additional reservoirs for the city water-supply. Mr. O'Shaughnessy has just reported to the Mayor and Common Council, recommending the construction of two new dams to provide reservoir capacity for future water-works needs in the city. The city at present owns sites for both dams and some preliminary work toward the utilization of the reservoirs in question has been done.

One of the two new dams proposed is at the mouth of the Barrett reservoir that lies just west of the old Morena reservoir. Here a reservoir with 7,000,000,000 gal. capacity will more than conserve 7,000,000 gal. daily average waste now taking place from lack of adequate storage. This would be, according to the consulting engineer, the cheapest stored water than can be added to the San Diego supply. In addition to the Barrett dam, Mr. O'Shaughnessy recommends a new dam on the Lower Otay, which besides collecting water from its adjacent watersheds can also be used to hold some overflow from Cottonwood Creek as may be diverted by the Dulzura conduit, which runs between the Barrett reservoir and Dulzura Creek, which latter empties into the Lower Otay reservoir.

If the city does not care to go to work on both dams immediately, the engineer recommends that the Barrett dam be given preference. The reservoir behind this dam would act not only as a balancing unit for the Lower Otay reservoir, but would also make full use of the completed Dulzura conduit and bring into immediate service the water now wasting on the Cottonwood, thus supplying a very desirable link in the ultimate plan of development.



CO. ON THE HOUSTON SHIP CANAL. OYSTER SHELLS BROUGHT IN ON TRESTLE AT RIGHT

With these two dams built, a much more complete utilization of the two watersheds now in the city's ownership will be possible and the available city supply will be increased in average years by approximately 10,000,000 gal. per day.

The data on the two dams may be recorded as shown in the accompanying table. It will be noticed that both

TABLE GIVING DATA ON BARRETT AND LOWER OTAY PROPOSED DAMS AT SAN DIEGO, CALIF.

	Barrett	Lower Otay
Height of overflow section above stream bed (ft.)	140	137.5
Height of nonoverflow section above stream bed (ft.)	152	145
Available storage at overflow (billion gallons)	6.75*	15†
Length of crest (ft.)	640	750
Length of available spillway crest (ft.)	242	212
Width of crest and roadway (ft.)	15	15
Depth of overflow (ft.)	12	75
Capacity of spillway (cu.ft. per sec.)	30,000	13,000‡
Volume of dam (cu.yd.)	89,000	86,000
Cost (estimated) (dollars)	690,400	682,200

* Can be increased to 7 billion gals. by flashboards.
† Can be increased to 15.5 billion gals. by flashboards.
‡ Additional spillway provided which will bring capacity up to 26,000 cu.ft. per sec.

dams are to be of the gravity section in arch plan, and are to be built of cyclopean or rubble concrete. Mr. O'Shaughnessy says that the exploration work of the two water-sheds has proved that the foundations are amply able to withstand all pressures from floods which may develop. These pressures should not exceed 10 to 11 tons per square foot for either structure. For both dams there is a central part made lower than the remaining in order to pass flood waters. It is estimated that the probable time for constructing the dams will be from 15 to 18 months if they are carried on simultaneously; if built in sequence from 30 to 36 months.

It is yet to be determined whether the people will be asked to vote for the construction of both dams at present, whether they will be allowed to choose between the two, or whether one of the two will be submitted to them for ballot. The city attorney has been instructed to outline preliminary steps for a bond issue.

Overtopped Earth Dam Fails

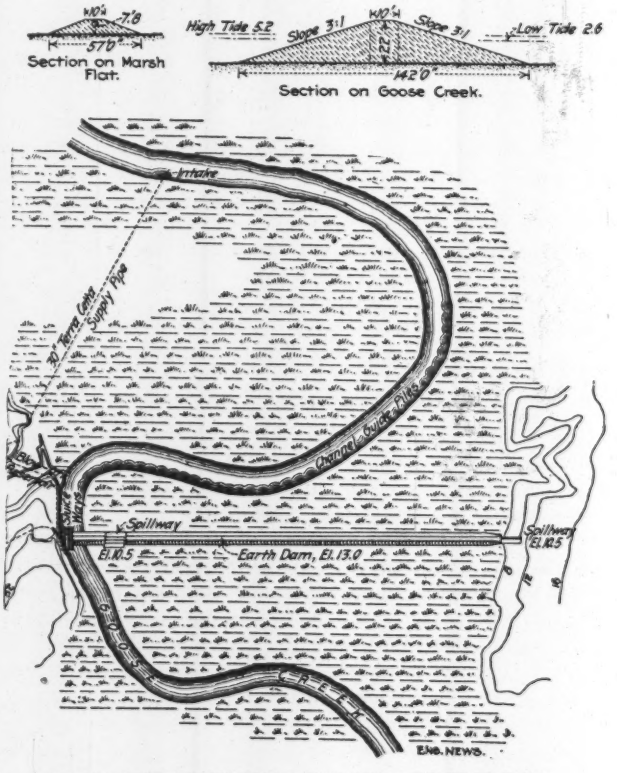
The partial failure by overtopping of the low earth dam of the Charleston (S. C.) Light and Water Co. on July 14 possesses interest beyond that due to the resulting damage. More than 16 in. of rain fell during the 24 hr. ending about the time the dam failed. The embankment rested on a marsh and when constructed 13 years ago required three times the clay fill indicated by the theoretical cross-section—on account of the way the fill sunk into the mud. The dam served the double purpose of impounding fresh water and shutting out salt water due to the tidal flow. Last to be mentioned, but by no means least, the dam was overtopped nearly four years ago; and some two years ago John H. Gregory, consulting engineer, New York City, reported (according to an editorial in a Charleston newspaper) in connection with valuation proceedings that the "dam was not a safe structure either under normal or abnormal conditions." As stated farther on, "legal complications" had a part in the postponement of action to make the dam safe.

Before taking up the failure and the emergency repairs that were rushed through to save the city from salt water or no water at all, some facts regarding the dam may be presented.[1]

[1]These facts, with the accompanying plan and sections, are taken from a long paper on "The New Water-Supply for Charleston, S. C.," read in 1906 by J. W. Ledoux before the Engineers' Club of Philadelphia and abstracted in "Engineering News," June 7, 1906, pp. 630-640.

The dam was completed in 1903 by the American Pipe Manufacturing Co., of Philadelphia, with J. W. Ledoux as chief engineer.

The marsh at the dam site consisted of an "alluvial deposit" of a "peaty nature," having an average depth of 9 ft. This was underlain with 10 ft. or more of "fine sand lying on a marly deposit." Goose Creek at the dam site was about 80 ft. wide and had an extreme depth of 20 ft. The total length of the dam was about 2,300 ft. Each of the two spillways shown on the plan was 100 ft. long, had a normal water depth of 2½ ft. and was built of timber cribwork, with sheet piling "extending down to solid material." The dam was built of "excellent material,



PLAN AND SECTIONS OF GOOSE CREEK DAM

principally composed of clay," dumped from cars running on a track extending the length of the dam. Drill holes sunk after the dam was completed "showed the clay fill to extend down to a depth of about 22 ft., or some 15 ft. below the marsh level." Knot grass was used to protect the slopes of the dam. Owing to wave action, Mr. Ledoux stated in 1906, this protection was insufficient on the upstream slope, "and hence it is proposed to cover this slope with a layer of granite riprap." Apparently, this was never done.

Under date of July 25, 1916, J. W. Ledoux, chief engineer of the American Pipe and Construction Co., 112 North Broad St., Philadelphia (successor to the company that built the dam) sent *Engineering News* an account of the rainfall that preceded the failure of the dam, together with a description of the failure and of the emergency repairs. Mr. Ledoux's notes except for a few sentences duplicating information taken from his article, as cited, follow:

On Friday, July 14, the rain gage at the pumping station [located close by the dam.—Editor] showed a rainfall of

3½ in. during the 24 hr., and this followed a rainfall of over 2 in. during the early part of the week. During the 24 hr. from Friday night to Saturday night there fell something over 16.3 in., the exact amount being unknown, as the gage was overflowed. According to observations made by a contractor living in the vicinity, the intensity of the rainfall at one time during Saturday was at the rate of 2 in. in 10 min.

Early on Saturday the water began to overflow the spillways and run over the earth dam. Dr. J. M. Worthen, superintendent in charge of the plant, did everything possible to prevent the erosion of the earth embankment, but at 7:30 Saturday evening, when the water was at the highest point and still rising rapidly and had a depth of about 14 in. over the earth dam, the downstream slope began to erode fast, and within half an hour a breach was washed clear across the dam, which at that point was approximately 50 ft. wide. The breach was about 100 ft. wide on top and approximately 13½ ft. deep. The top of the embankment was at El. 13.5 above ocean tide. The bottom of the breach was approximately zero. The crest of the spillways was 10½ ft. above tide, and at that point the capacity of the dam was 2,800,000,000 gal.

On Monday morning, July 17, the elevation of the water was 8 ft. above tide, and on Tuesday morning 7 ft. The capacity of the dam at El. 7 was 782,000,000 gal. Ordinarily, the water in Goose Creek downstream from the dam ranges in elevation from zero to 6 ft. above tide, but during the heavy rainfall, just before the dam broke, the elevation of the water below the dam was approximately 10 ft. above tide.

The storm was very intense; telephone and telegraph lines were down, and streams were very much swollen between Goose Creek pumping station and Charleston, so it was difficult to get word into the main office concerning the danger. Finally they succeeded in sending a messenger, and a pile-driving plant and other apparatus were shipped out over the railroad. The water company succeeded in getting a wrecking crew from the Seaboard Air Line, which worked continuously to make such repairs as would prevent the city from getting out of water. This work was directly under the personal charge of W. R. Bonsal, Vice-President of the Seaboard Air Line. The water company was fortunate to have such an able crew of riggers, wreckers and rivermen under such efficient management.

The first work consisted in a coffer-dam composed of sandbags laid at the upstream end of the breach, where the depth of the water ranged from 4 to 9 ft. This coffer-dam was started Tuesday morning, July 18, and reached the opposite side of the creek at 3 a.m., July 19. At that time the water was rising at the rate of 1½ in. per hr. Fortunately the high tide occurred about the time of completion, so that there was little current. However, by the next morning at 8 o'clock, when the tide had receded 2 ft., the velocity over the coffer-dam was so great that the sandbags began to show signs of displacement. For half an hour it looked as if the whole coffer-dam would fail. The expedient was then adopted of covering the worst places of the coffer-dam with canvas, weighting it down on the upstream side and letting it lap entirely over the structure. This proved so successful that it was decided to cover the whole coffer-dam with canvas in the same way.

When finished, the bottom of the coffer-dam was approximately 25 ft. wide, the top 6 ft. and the length about 120 ft. It has stood successfully a flow of water over the top of about 6 in., which was sufficient at that time to take care of the flow of the creek.

Considerable fear was felt among the inhabitants of Charleston concerning the continuance of the supply of water, and it was thought the water company would have difficulty in preventing the salt tide from backing up into the impounding reservoir, but on account of the quick work this was safely prevented.

Permanent repairs are now being made by means of heavy piling driven clear across the breach and on both sides of same. These piles were driven on 8-ft. centers and 8 ft. apart in the direction of the stream. Four-inch sheet piling was driven on each side, and the space within and on each side filled with sandbags, after which the earth filling will be completed.

The maximum flow of the stream must have been nearly 20,000 cu.ft. per sec. when the break took place. The remainder of the embankment was very little damaged, although the water had run over the entire structure to a depth of over 1 ft. for a considerable time.

For several years it has been recognized that [the two 100-ft.—Editor] spillways were inadequate to carry off flood flows, as their discharging capacity was only 3,500 cu.ft. per sec., or 70 cu.ft. per sec. per sq.mi.

On Sept. 13, 1912, the water ran over the dam, which was then at El. 12.34, to a depth of approximately 0.7 ft., without causing any damage. It was then decided to construct an adequate concrete spillway, but for legal and other reasons, well known to the parties most interested, it has been up to date impracticable to do this, but it is hoped now that immediate steps will be taken to carry out such an essential improvement.

Cerro Prieta Irrigation Canal

The Imperial Irrigation District is building a canal 22 mi. long, 60 ft. wide and 8 ft. deep as an emergency project, under the direction of C. R. Rockwood, Chief Engineer, Calexico, Calif. The canal will head in a gate placed in the levee of Volcano Lake and will extend along the west side of New River. The lake is filled by overflowage from the Colorado River. This section of the Cerro Prieta canal as it is called, is designed to prevent a water shortage in August, due to the silting up of the upper four miles of the main canal and the resulting possibility that the district may be unable to obtain a sufficient supply of water through the Alamo canal during the months of August and September.

Mr. Rockwood says further regarding this project:

It is the intention this winter to extend this canal from the Cerro Prieta gate, at the west end of the levee, eastward along the levee to a connection with our Alamo canal, at a point known as the Cudahy Check. This will permit us during the low-water season, if we have water to spare, to continuously draw a large volume through the Alamo canal from the river and carry it through the Volcano Lake, and by that means not only deposit the silt from the river along the Volcano Lake levee where it is necessary for our protection, but at the same time it will permit us to deliver to all lands on the west side of New River, water from which the heavy silts have been removed, thereby very greatly decreasing our cost of maintenance.

This canal, when finally completed, will become a very important part of our distribution system, as it will permit us to eventually do away with the flume across New River and will also relieve the Alamo canal of its overload for the lower 20 miles.

The entire canal construction now in progress is in earth that is being moved with Fresno scrapers. In July there were 1,700 mules on the work. The work was then being prosecuted with great difficulty on account of the heat and also because the unsettled conditions across the border line in Mexico, where the work is being carried out, make it very difficult to obtain labor.

A Serious Shortage of Electric Power occasioned a temporary stoppage in the operation of the Ontario Hydro-Electric system on July 17, affecting 140 municipalities on the Niagara circuit. This has been anticipated for some time, owing to the steadily increasing demands of the municipalities dependent on the system. Sir Adam Beck, chairman of the Commission, states that there is an actual present shortage of 25,000 hp., and that before the end of the year 50,000 hp. will be absolutely necessary if the requirements of the public are to be met. The Ontario Government, on being appealed to to secure additional power from development companies which now export it to the United States, referred the matter to the Canadian Government which has the power of stopping the exportation. The power companies at Niagara Falls are bound to supply the Commission with half of their output at the price obtainable in the American market. The Canadian Niagara Power Co., which sends its entire output of 75,000 hp. to New York State, receives therefor $18 to $20 per hp., which is double the price the Hydro-Electric Commission has been paying. They offered the Commission 35,000 hp. at American rates, but the proposition could not be accepted, as it would have meant a proportionate increase in the price charged to the municipalities. Pressure was brought to bear upon the company by the Canadian Government, and rather than incur the risk of having their export trade cut off the Canadian Niagara Power Co., has agreed to furnish the Hydro-Electric Commission with 12,500 hp. immediately, 50,000 hp. by the end of the year and 100,000 hp. if required thereafter at $12 per hp.

News of the Engineering World

Detroit-Superior Viaduct at Cleveland Makes Progress

Visibly rapid progress is being made this season on the concrete-viaduct portion of the Detroit-Superior bridge across the Cuyahoga River in Cleveland, Ohio. A recent view is reproduced herewith. The portion nearest the 591-ft. steel river arch still lacks the deck construction. Completion is expected this year. The Hunkin-Conkey Construction Co. is building the concrete viaducts on both banks.

The original plan for carrying the street cars from the lower deck to street level by inclines on the approaches was abandoned last year. Street-car subways are to be built for a length of several blocks beyond either end of the bridge.

Several remarkable phenomena marked the progress of the concrete work. The plan of operations necessarily involved the omission of the concrete arch adjoining the steel river arch while the steelwork was under erection. Therefore the placing of the landward concrete arches produced unbalanced thrusts on a series of piers, which resulted in some longitudinal deflection of the piers, either elastic or permanent. No serious effects of this deflection are known. Within the past two weeks, however, a crack developed in one of the four arch ribs of one span. This is now being cut out and concreted, after reducing the rib to a no-stress condition by jacking up on falsework. A. W. Zesiger, county bridge engineer, is directing the work, under W. A. Stinchcomb, county surveyor. K. D. Cowen is resident engineer.

Engineer Becomes President of New Bridge Company

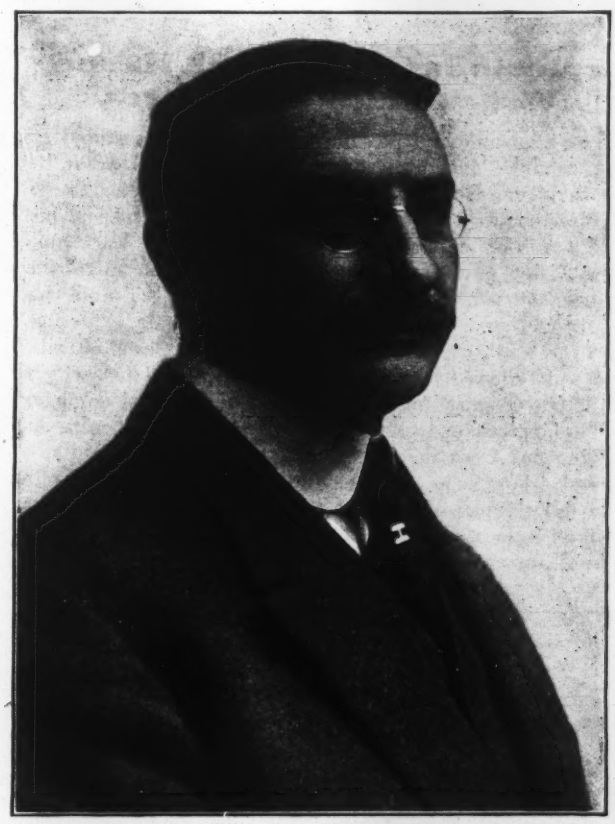
As announced in the last issue of *Engineering News*, George H. Blakeley has been made President of the newly organized Bethlehem Steel Bridge Co., which is to carry on the structural business of the Bethlehem Steel Co. and the business formerly conducted by the bridge department of the Pennsylvania Steel Co. Mr. Blakeley has long been a prominent figure in the field of structural engineering and his appointment as the executive head of this new organization is a notable recognition of the ability of the technical engineer to fill positions of high responsibility.

Mr. Blakeley became widely known to the profession as Chief Engineer of the Passaic Rolling Mill Co. of Paterson, N. J., where he carried the chief engineering responsibility for fifteen years. He graduated from the Scientific Department of Rutgers College in 1884. After two years' work in various lines of field engineering he engaged in bridge construction, first with the Riverside Bridge Co., of Paterson, and later as assistant engineer

DETROIT-SUPERIOR BRIDGE AT CLEVELAND IS NEARING COMPLETION
View of south side of east approach. Upper deck of three concrete arches uncompleted.

of the Erie R.R. in charge of bridge construction under the late C. W. Buchholz, then Chief Engineer. He was appointed Chief Engineer of the Passaic Rolling Mill Co. in 1890, and during his connection with that company he had charge of the building of many important structures. Among them was the Harlem River Bridge at 155th St., of which the late Alfred P. Boller was consulting engineer. This bridge at the time it was built was the heaviest swing span in existence, and the first to have a double drum and double ring of bearing wheels.

In 1893 Mr. Buchholz invited Mr. Blakeley to come back to the Erie as Assistant Chief Engineer, but he preferred to continue in his work in structural engineering. In 1897 the Passaic company issued a structural steel handbook compiled by Mr. Blakeley, containing a large amount of original information on structural steel. This received a hearty welcome from the engineering profession, and the precedent set by it was later followed by other steel companies in the preparation of their handbooks.



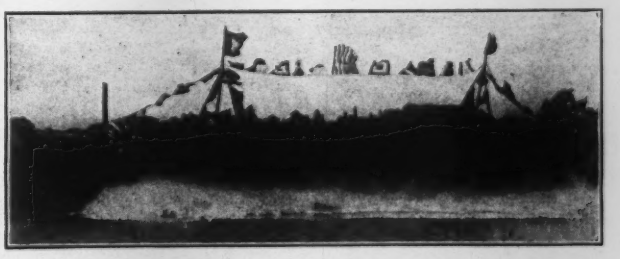
The years Mr. Blakeley remained with the Passaic company were years of great development in American bridge engineering. The typical American pin-connected truss of the '80's was replaced by riveted structures of modern type with carefully designed details. During this period also, steel-frame construction for modern buildings had its development. Mr. Blakeley was a prominent figure in both these fields of progress.

It is the rule, rather than the exception, for civil engineers to stick closely to their specialties and fight shy of work in other fields of engineering. Mr. Blakeley, however, while with the Passaic company, was placed in charge of the mechanical engineering work in connection with the company's steel mills, and made a thorough study of rolling-mill methods. In 1902 the late Henry Grey returned to the United States after his successful work abroad in placing in operation the first Grey mill for rolling heavy steel beams at Differdingen. Mr. Blakeley came in touch with Mr. Grey and took a lively interest in the Grey mill and its possibilities for producing heavy structural sections. In 1906 Mr. Charles M. Schwab started to develop the Grey mill process in the United States at the Bethlehem Steel Co.'s plant, and Mr. Blakeley severed his connection with the Passaic Rolling Mill Co. and became connected with this enterprise. For the following two years Mr. Blakeley and Mr. Grey were engaged in the work of building the mill and studying the problem of the sections most available for structural purposes, the result being the development of the broad flange "Bethlehem" sections now so widely used. The mill was put in operation in 1908, and Mr. Blakeley was made Manager of Structural Steel Sales of the Bethlehem Steel Co.

With the acquisition of the Pennsylvania Steel Co. by the Bethlehem company, the responsibility for the structural work of that organization is now added to Mr. Blakeley's former tasks. Mr. Blakeley is a director of the Bethlehem Steel Co., a member of the American Society of Civil Engineers, of the American Society of Mechanical Engineers, and of the Engineers' Club of New York City.

Launch of a Survey Steamer

The steamship "Surveyor" for the U. S. Coast and Geodetic Survey was launched July 22 at the plant of the Manitowoc Ship Building Co., Manitowoc, Wis., in the presence of Dr. E. Lester Jones, Superintendent of the Survey, whose daughter Elizabeth performed the christening ceremony. The ship was launched sideways, in accordance with general practice at lake shipbuilding yards, and the accompanying view shows it entering the water.



LAUNCHING THE NEW SURVEY STEAMER

It is a steel vessel 186 ft. long, 34 ft. beam and 11½ ft. draft, with a displacement of about 1,000 tons. It has a triple-expansion engine of 1,000 hp., and will use oil fuel. The officers and crew will number about 66, and the vessel will carry enough fuel and supplies for a 3-months' cruise.

Although the vessel was built on the Great Lakes it is intended for service in the Pacific, surveying and charting the Alaska Coast and Behring Sea during the summer, and the Pacific Coast during the winter. It is expected to have the ship finished this fall and sent through the lakes and the St. Lawrence to the Atlantic before the navigation season is closed. The contract price for the hull and machinery was $189,000, but when fully equipped the vessel will represent a cost of about $200,000.

Cleveland Tunnel Accident Not Yet Explained

The Cleveland intake-tunnel accident of Monday, July 24, 1916, has not yet been investigated fully enough to give any real information as to its nature or origin. At the time of going to press no one had penetrated to the end of the heading driven landward from the outer crib. That an explosion occurred is considered proved by the distorted and disordered condition of rails and cars, etc., so far as the search parties had advanced. No explosive was used in the tunnel. Just two days before the accident, on Saturday evening or Sunday morning, trouble with gas had been reported by the last shift of the week, on coming out; the men said they felt the effects of gas. So far as now known no analysis of the air was made. The system of working required, in case gas was noticed, using such ventilation as could be secured by blowing through the 8-in. compressed-air supply pipe and drawing off air through the 4-in. blowpipe or through the airlock valves, before the next shift entered. No shift went in Monday morning, after the Sunday stop. The Monday night shift was taken in by an assistant engineer (who had previously himself worked in the heading gang), acting as heading foreman, in the absence of the regular foreman.

As noted last week, the superintendent of the tunnel work enforced strict rules against any use of matches, open lights, photographers' flashlights or other source of flame, in both land and crib headings. The occasional occurrence of gas was known, and its dangers were recognized from earlier experiences in tunneling in the same ground. On the other hand there was no regular check kept on gas occurrence, chemical analyses being made only occasionally.

The nature of the clay in which the tunnel is located makes it probable that damage to the tunnel at the face will be found to be slight.

Explosions in New York Harbor Wreck Many Structures

Two tremendous explosions in the munitions stored at and around a freight terminal in New York Harbor in the early morning of July 30, completely wrecked most of the structures in the immediate vicinity, broke about half a million dollars worth of glass in New York and New Jersey buildings, caused the death of an unknown number of persons (listed as six on Aug. 2) and the injury of a great number of others. The main explosion at 2:08 a.m., was felt over a radius of at least 50 mi., and was followed by a second about 30 min. later, which while not of equal force caused most of the local damage.

The explosions took place on Black Tom, a peninsula jutting out from the New Jersey shore just off Bedloe's Island, where is located the Statue of Liberty, and about two miles south of west of the Battery. On this point of land is located a large terminal, consisting of a number of warehouses fed by the Lehigh Valley R.R. and

SOME OF THE WRECKED WAREHOUSES ON BLACK TOM IN NEW YORK HARBOR

many piers from which are loaded the barges and lighters used in harbor transhipment. About midnight on July 29 a fire broke out either in one of the barges or in a pier shed or waiting freight car. The Jersey City fire department was called out to the fire but found it beyond control among the barges and freight cars, many of which were loaded with munitions awaiting shipment to the European battlefront. One of the barges was being towed away from the pier when it exploded, at 2:08 a.m., causing the concussion which shook up the whole surrounding territory and scattered the burning débris all over the peninsula. The fires so set reached a train containing high explosives which blew up about 2:30 a.m., and wrecked most of the structures in the terminal. Some of this local damage is shown in the accompanying view. The fire spread by this explosion added to the destruction.

Subsequent damage was caused by exploding shells on the drifting and burning barges, which continued until well beyond daylight.

The money value of the damage is not yet known, but is estimated at from $20,000,000 to twice that amount. Responsibility for the storage of so much high explosive near the city—contrary to law—is also not fixed, but the usual multiplicity of investigations is assured.

Earth Fill Coffer-Dam Fails at Boston Dry Dock

About 100 ft. of the coffer-dam at the new South Boston dry dock failed at high tide during the night of July 25. The site had been excavated by dredging before the coffer-dam was closed, and the section across the outer end, some 500 ft. long, had been only recently completed. Pumping had been begun and the level of the water had been reduced to about 8.5 ft. below low tide when work stopped on the afternoon of July 25. The coffer-dam gave way during the night under a head of about 18.5 ft., when no one was working in the basin, so that no damage was done except to the structure itself.

The coffer-dam consists of two lines of 6-in. sheetpiling, supported by round piles and spurshores, cross-braced and tied inside, filled with puddle, and banked up outside with earth and riprap. At this location the harbor silt is underlain by a stratum of hard blue clay, resting on soft material which overlies the rock. The coffer-dam piling is driven about 5 ft. into the blue clay.

All ordinary precautions had been taken to make the structure stable, and the design was one which had been used with entire success elsewhere about Boston. The reason for the failure is therefore not clear. At the present time the entire section of the coffer-dam carried out is below low water level, so that it cannot be readily examined. The work is being done by the Holbrook, Cabot and Rollins Corporation as contractors working heretofore for the Directors of the Port of Boston, one of the commissions which recently has been superseded by the new Massachusetts Commission on Waterways and Public Lands. No attempt will be made at the present time to pull up the old coffer-dam and a new one will be built outside of it.

A Municipal Electric Lighting Plant for St. Louis is planned by A. M. Branch, Commissioner of Light, Heat and Power. Investigations to determine its feasibility are to be undertaken.

Bids for Water-Power Rights at the dam at Joliet, Ill., have been received by the Illinois and Michigan Canal Commission at Springfield, Ill., the lease held by the Economy Light and Power Co. (a subsidiary of the Chicago Edison Co.) having expired. The Sanitary District of Chicago bid $15.01 per hp. per year and Andrew Cooke bid $3; the latter is understood to have represented the Economy company. It is stated that if its bid is accepted the Sanitary District will be able to install 10,000 additional street lights in Chicago and that it proposes to double the present 3,000-hp. development.

Boston Paving Squabbles—In order to obtain the kind of pavement which they want, residents of Commonwealth Ave. in Boston have paid to the city the difference between the prices bid for bitulithic and for sheet asphalt. A short time ago bids were received by the Public Works Department for paving Commonwealth Ave. with sheet asphalt, Topeka pavement, or bitulithic. The bid for the sheet asphalt was the lowest, and the mayor had decided to accept it, when the Commonwealth Ave. residents took up a collection amounting to $7,907.90 and sent a check for that amount to the mayor. He at once signed a contract with Warren Brothers Co. for the bitulithic pavement which the abutters desire.

The Waterways and Public Lands Commission of Massachusetts, a new institution which supersedes and will handle the work of the old Harbor and Land Commission and the Directors of the Port of Boston, has been named by Governor McCall. The members of the new commission are: John M. Cole, Chairman, long prominent in state politics as Speaker of the House of Representatives, Chairman of the State Board of Efficiency and of the Boston Industrial Development Board; Jesse B. Baxter, also a prominent politician, a member of the Republican State Committee, and William S. McNary, who has been Chairman of the Harbor and Land Commission for several years. Their terms are three, two and one years respectively.

The Engineers Club of Philadelphia has set a high mark in the issue of its "Proceedings" for July. The size of the publication has been enlarged to 9x12 in. and it contains no less than 112 pages of matter including advertisements. The club has carried through with remarkable success a broad campaign for increasing its own membership, which has raised its list from 500 to nearly 2,500, and a plan under which seven local sections of national engineering societies, with a resident membership of some 1,500 engineers, have affiliated with the club. Each of these seven sections has its own department in the "Proceedings" in which to present matter of special interest to its members, including records of its current meetings. The "Proceedings" are conducted by the Publication Committee of the club, with Joseph Caccavajo as Editor-in-Chief.

New Pittsburgh Terminal Plans—The plans which were announced some time ago ("Engineering News," March 23, 1916) for the reconstruction of the Pennsylvania R.R. station in Pittsburgh to provide largely increased office space have now been modified. It is reported that the railroad is working on a plan that would bring the station three blocks nearer the center of the business district, and which would involve rather extensive street changes. Grant St. would be extended through to Liberty Ave. in a tunnel under the Pennsylvania tracks, and the present narrow alley known as Cherry Way would be widened 30 or 40 ft. to give a second main thoroughfare across town to the outer Penn Ave. district. The new station, according to this plan, would be located on the very edge of the district improved by the cutting of the Pittsburgh "Hump."

New Belle Isle Bridge—Acting on the suggestion of George H. Fenkell, Commissioner of Public Works, Detroit, Mich., Mayor Marx has appointed a consulting board to assist the Commissioner of Public Works in the preparation of plans and estimates for construction of a new bridge to connect Belle Isle with the mainland. This board, which serves without compensation, consists of the following: Mortimer E. Cooley, Dean of the College of Engineering and Architecture, University of Michigan; William R. Kales, Civil Engineer, Detroit; Francis M. McMath, Civil Engineer, Detroit; Emil Lorch, Professor of Architecture, University of Michigan; Henry E. Riggs, Professor of Engineering, University of Michigan. Offices have been opened at 34-36 West Congress St. The Belle Isle bridge was destroyed by fire Apr. 29, 1915 (see "Engineering News," May 6, 1915). A temporary timber structure in place of this bridge is now under construction.

Closer Relations between the National Engineering Societies and the National Academy of Sciences have been under discussion for some time and a joint committee has now been appointed to consider plans for coöperation in any work of mutual interest. The representatives appointed on this joint committee are as follows: American Society of Civil Engi-

neers, Gen. Wm. H. Bixby; American Institute of Mining Engineers, Sydney J. Jennings, New York City; American Society of Mechanical Engineers, Ira N. Hollis, Worcester, Mass.; American Chemical Society, Dr. L. H. Baekeland, Yonkers, N. Y.; American Institute of Electrical Engineers, Dr. J. J. Carty, New York City; Engineering Foundation, Gano Dunn, New York City. Representatives of the National Academy of Sciences are Jos. S. Ames, Johns Hopkins University; John F. Hayford, Northwestern University; Edward L. Nichols, Cornell University; M. I. Pupin, Columbia University; E. B. Rosa, Bureau of Standards; Elihu Thomson, Lynn, Mass.; C. R. Van Hise, University of Wisconsin; Chas. D. Walcott, Smithsonian Institution; Robert S. Woodward, Carnegie Institute; George Ellery Hale, Chairman, Pasadena, Calif.

Nashville Water-Works Superintendent—A legal fight of interest to water-works men is the successful suit of George Reyer, former superintendent of water-works of Nashville, Tenn., for reinstatement. By a resolution adopted in 1913 the Board of Public Works abolished the office, the resolution stating that the failure of the reservoir wall in November, 1912 (see "Engineering News," Nov. 14, 1912), had been due to the superintendent's negligence. Mr. Reyer contended that the abolition of the office was merely a desire to oust him. The case was carried to the court of civil appeals and on the second trial in the circuit court the city contended that the position of superintendent had been absorbed by the office of commissioner of water-works, street-cleaning and workhouse. This question was also carried to the higher court, which remanded it for a rehearing in the circuit court on the issue whether a separate water-works superintendent was necessary or not. The charter of the commission-form government provides for a water-works superintendent, and the reports of the National Board of Fire Underwriters and of James Cameron and H. M. Brinkerhoff on an efficiency survey of the municipal government, both urge strongly the appointment of a technically trained water-works superintendent. Mr. Reyer's record as a water-works superintendent was put in as evidence, and he contended that the position required the services of a man of such technical training. The circuit court ordered Mr. Reyer reinstated with accrued salary. The Board of City Commissioners has voted to acquiesce in the decision of the court without appealing the case, and Mr. Reyer was reinstated Aug. 1 with back salary to May, 1913. Following the publication of reports that an appeal would be taken, the Nashville Section of the Engineering Association of the South, at a special meeting, adopted a resolution declaring that the welfare of the city demanded that a competent engineer be appointed superintendent of the water-works immediately.

Apportionment of Federal-Aid Road Funds—On July 21 the Secretary of Agriculture announced the apportionment of the $5,000,000 Federal-aid road appropriation as follows:

Alabama	$104,148.90	Nevada	$64,398.30
Arizona	68,513.52	New Hampshire	20,996.62
Arkansas	82,689.10	New Jersey	59,212.68
California	151,063.92	New Mexico	78,737.81
Colorado	83,690.14	New York	250,720.27
Connecticut	31,090.44	North Carolina	114,381.92
Delaware	8,184.37	North Dakota	76,143.06
Florida	55,976.27	Ohio	186,905.42
Georgia	134,329.48	Oklahoma	115,139.00
Idaho	60,463.50	Oregon	78,687.37
Illinois	220,926.23	Pennsylvania	230,644.17
Indiana	135,747.62	Rhode Island	11,665.71
Iowa	146,175.60	South Carolina	71,807.64
Kansas	143,207.40	South Dakota	80,946.02
Kentucky	97,471.91	Tennessee	114,153.48
Louisiana	67,474.66	Texas	291,927.81
Maine	48,451.50	Utah	56,950.15
Maryland	44,047.22	Vermont	22,844.47
Massachusetts	73,850.95	Virginia	99,660.71
Michigan	145,783.72	Washington	71,884.28
Minnesota	142,394.06	West Virginia	53,270.46
Mississippi	88,905.84	Wisconsin	128,361.07
Missouri	169,720.41	Wyoming	61,196.82
Montana	98,287.19		
Nebraska	106,770.81	Total	$4,850,000.00

These funds are available for the fiscal year ending June 30, 1917. The sum of $150,000 or 3% is deducted for the expenses of administration, that is, for the United States Office of Public Roads and Rural Engineering. The remaining sum is apportioned among the states, one-third in ratio of area, one-third in ratio of population and one-third in ratio of mileage of rural mail delivery and star routes.

The Committee on Industrial Preparedness of the Naval Consulting Board, through Howard E. Coffin, its Chairman, announces that it has nearly completed the industrial inventory of the United States which has been made through the coöperation of members of the National Engineering Societies and has been in progress during the last three months. The inventory covers more than 30,000 manufacturing plants in the United States which do a business of over $100,000 a year, besides many smaller plants producing equipment especially useful in time of war. The committee further states that legislation will probably pass Congress during the present session to carry out the entire program of the committee, including the plan for training manufacturing plants in the production of war munitions by the distribution of annual "educational orders" which will enable works to have actual experience in the processes necessary to produce munitions. Legislation is also hoped for providing for the organization of an "industrial reserve," so that skilled mechanics, toolmakers, etc., may be kept at work in the shops in the event of war instead of taken away for service in the fighting line—a matter of the utmost importance to a nation's military efficiency. The necessity of legislation by Congress is apparent from a statement by Mr. Coffin, made in a recent address before the Engineers Club of Philadelphia, to the effect that under a law enacted Feb. 27, 1906, the United States Government cannot accept voluntary service from persons not in the Government's employ. The language of the statute is:

> Nor shall any department or any officer of the Government accept voluntary service for the Government, or employ personal service in excess of that authorized by law, except in case of sudden emergency involving the loss of human life or the destruction of property.

A way to give authority to the Committee on Industrial Preparedness, however, has been opened by the passage of the National Defense Bill on June 3. In this bill the Secretary of War was authorized to cause to be made a complete list of all private plants in the United States equipped to manufacture munitions and the President was authorized to appoint a board on mobilization of industries essential for military preparedness.

PERSONALS

L. R. Talbot has been appointed Construction Engineer of the Shawinigan Water and Power Co., Shawinigan Falls, Que.

S. M. Bate has been appointed Engineer of the Kansas City Stock Yards Co. to succeed Henry A. Wise, resigned, as noted elsewhere.

E. Y. Smith, Manager and Treasurer of the Florida Pile and Timber Co., Jacksonville, Fla., has been placed in charge of the company's plant at Natchez, Miss.

Melbert W. Taber, M. Am. Soc. M. E., formerly Construction Engineer, Packard Motor Car Co., has been made Manager of the Detroit office of the Asbestos Protected Metal Co. of Pittsburgh.

William G. Sloan, M. Am. Soc. C. E., chief engineer of MacArthur Brothers Co., general contractors, 120 Broadway, New York City, has been elected vice-president to succeed F. C. Hitchock, resigned, as noted elsewhere.

Samuel Murray, Assoc. M. Am. Soc. C. E., Acting Chief Engineer of the Oregon-Washington Railroad and Navigation Co., Portland, Ore., since the retirement of J. R. Holman in September, 1915, has been appointed Chief Engineer.

Francis R. Shunk, Lieutenant-Colonel, Corps of Engineers, U. S. A., formerly in charge of the Pittsburgh district, has been made Resident Member of the Rivers and Harbors Board, Washington, D. C., succeeding Col. Henry C. Newcomer.

Howard W. Morgan, recently Senior Engineer with the Sewerage Commission of the City of Milwaukee, Wis., has become associated with the Dorr Co., Engineers, 17 Battery Place, New York City, and will specialize in sewage disposal problems.

N. H. Bogue is Chief Engineer of the proposed Oregon, California & Eastern Ry., which is being promoted by Robert E. Strahorn, former Vice-President of the Oregon-Washington Co. The general offices of the company are in Portland, Ore.

George A. Ricker, M. Am. Soc. C. E., Consulting Engineer, of Albany, N. Y., has accepted a position with the Portland Cement Association in charge of its Kansas City office. Mr. Ricker is a former Deputy Highway Commissioner of New York State.

E. G. Lane, Assoc. M. Am. Soc. C. E., formerly District Engineer, Maintenance-of-Way, Baltimore & Ohio Southwestern R.R., Cincinnati, Ohio, has been promoted to be Engineer of Maintenance-of-Way. His headquarters will remain at Cincinnati.

A. L. Hoerr, M. Am. Soc. M. E., former Steam and Hydraulic Engineer of the National Tube Co., Pittsburgh, Penn., has been appointed Chief Engineer of the National Works of the company. He is succeeded as Steam and Hydraulic Engineer by Frank E. Leahy.

Samuel J. Hoexter, Assoc. M. Am. Soc. M. E., Instructor in Mechanical Engineering at the University of Michigan, has been made Manager of the Engineers' Society of Pennsylva-

nia, Harrisburg, Penn. He is a graduate of Stevens Institute of Technology, class of 1909.

Edward P. Burch, F. Am. Inst. E. E., Consulting Engineer, Detroit, Mich., has been selected by Mayor Davis, of Cleveland, Ohio, to represent the city as electric lighting expert at a hearing before the state public utilities commission in support of a 3c. per kw.-hr. electric-lighting rate.

F. C. Hitchcock, M. Am. Soc. C. E., has resigned as vice-president and general manager of MacArthur Brothers Co., general contractors, 120 Broadway, New York City. He has become vice-president and general manager of the Siems Carey Railway and Canal Co., which has extensive work in China.

S. T. DeLaMater, M. Am. Soc. C. E., Consulting Engineer, Chicago, Ill., is now President of the Chicago General Construction Co., Manhattan Bldg., Chicago. He was previously with Graham, Burnham & Co., James Stewart & Co., L. P. and J. A. Smith and other well known engineering construction companies.

Henry A. Wise has resigned after 20 years' service as Engineer of the Kansas City Stock Yards Co. He became Engineer of the company in 1892 and in 1896 was made City Engineer of Kansas City, which position he held for four years. He returned to the Stock Yards Co. in 1900 and has been there ever since.

Henry C. Newcomer, Colonel, Corps of Engineers, U. S. A., formerly Resident Member of the River and Harbor Board at Washington, D. C., has been made Principal Assistant to Gen. William M. Black, Chief of Engineers, and has been placed in charge of the rivers and harbors division, relieving Col. Harry Taylor, who has been transferred to Governors Island, N. Y.

Stuart K. Knox, M. Am. Soc. C. E., Consulting Engineer, New York City, has recently become a member of the firm of Hill & Ferguson, Consulting Engineers, 100 William St., New York City. For the past eight years Mr. Knox has been associated with the senior member of the firm (Nicholas S. Hill, Jr.) as Principal Assistant Engineer. He is a graduate of the University of Michigan.

W. R. Armstrong, M. Am. Soc. C. E., General Manager and Chief Engineer of the Salt Lake & Utah R.R., Salt Lake City, Utah, has been appointed Engineer of Maintenance-of-Way of the Union Pacific R.R., with headquarters at Omaha, Neb. He was formerly Superintendent of the Montana division of the Union Pacific and has had about 25 years' experience in railway engineering and operation.

Michael Edward Brian, Assoc. M. Can. Soc. C. E., City Engineer of Windsor, Ont., has been made Chairman of the board of engineers which will prepare plans for joint water-supply and sewerage systems for Windsor, Walkerville, Ford City, Sandwich, Ojibway and Sandwich West. Plans will be prepared in time for a vote by the taxpayers of the several towns next January. Leonard Rice, Assistant Engineer, of Ojibway, is Secretary of the board.

John I. Mange has been elected vice-president of the J. G. White Management Corporation, New York City. Mr. Mange has been associated with the J. G. White Management Corporation since 1912, at which time he was elected vice-president of the Associated Gas and Electric Co., a holding company of electric and gas properties in New York, Kentucky, Ohio and Tennessee, managed by the J. G. White Management Corporation. He was born in Pembroke, Me., in 1876. He attended the public schools in Pembroke and Kingston, Mass., and was graduated in 1899 from Tufts College, with the degree of bachelor of science.

Joseph L. Nobre and **Richard D. McFadden,** who were dropped from the engineering forces of the Philadelphia Department of Public Works by former Director Morris L. Cooke in July, 1913, on charges of political activity, have been reinstated by Director George E. Datesman. Mr. McFadden has been connected with the Department of Wharves, Docks and Ferries as a state appointee. Both engineers will be given their old positions as District Surveyors of the Bureau of Surveys at salaries of $4,000 per annum. William H. Reeder and Charles F. Puff, who have held these offices since 1913, will be assigned to other work.

OBITUARY

Howard Macrum, General Superintendent of the Otis Steel Co., Cleveland, Ohio, died recently at his home in Cleveland, aged 43 years.

William Alexander Park, a civil engineer of Knoxville, Tenn., died July 19, aged 53 years. He was born in 1863. He was graduated from the University of Tennessee and spent the greater part of his life in the vicinity of Knoxville.

S. H. Volks, Assistant Engineer of Construction of the Cleveland, Ohio, Water Department, was among those who lost their lives in the tunnel accident, July 24, 1916. He was a 1911 graduate of the Case School of Applied Science.

William Rodney Patterson, M. Am. Soc. C. E., of the firm of Patterson & Davidson, Consulting Engineers, Chicago, Ill., died July 20. He was 62 years old and had been a resident of Chicago for 39 years. He was an inventor of lead covered electric cables.

ENGINEERING SOCIETIES

INTERNATIONAL ASSOCIATION OF FIRE ENGINEERS.
Aug. 29-Sept. 1. Convention in Providence, R. I. Secy., James McFall, Roanoke, Va.

ASSOCIATION OF EDISON ILLUMINATING COMPANIES.
Sept. 4-7. Convention in Hot Springs, Va. Asst. Secy., E. A. Baily, 360 Pearl St., Brooklyn, N. Y.

THE TRAVELING ENGINEERS' ASSOCIATION.
Sept. 5-8. Annual meeting in Chicago, Ill. Secy., W. O. Thompson, care General Offices, N. Y. C. R.R., Cleveland, Ohio.

AMERICAN FOUNDRYMEN'S ASSOCIATION.
Week of Sept. 11. Annual meeting in Cleveland, Ohio. Secy., A. O. Backert, 12th and Chestnut St., Cleveland, Ohio.

THE AMERICAN INSTITUTE OF METALS.
Week of Sept. 11. Meeting in Cleveland, Ohio. Secy., W. M. Corse, 106 Morris Ave., Buffalo, N. Y.

NATIONAL ASSOCIATION OF STATIONARY ENGINEERS.
Sept. 11-16. Convention, Minneapolis, Minn. Secy., Fred W. Raven, 417 S. Dearborn St., Chicago.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 12. Convention in Atlantic City. Secy., A. P. Dane, Reading, Mass.

RAILWAY SIGNAL ASSOCIATION.
Sept. 12-14. Convention in Mackinac Island. Secy., C. C. Rosenburg, Myers Building, Bethlehem, Penn.

NEW ENGLAND WATER-WORKS ASSOCIATION.
Sept. 13-15. Convention in Portland, Me. Secy., Willard Kent, Narragansett Pier, R. I.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 18-20. Convention in Philadelphia, Penn. Asst. Secy., C. D. Fawcett, 29 West 39th St., New York, N. Y.

ASSOCIATION OF IRON AND STEEL ELECTRICAL ENGINEERS.
Sept. 18-22. Convention in Chicago. Secy., W. O. Oschmann, Oliver Iron and Steel Co., Pittsburgh, Penn.

ROADMASTERS AND MAINTENANCE OF WAY ASSOCIATION OF AMERICA.
Sept. 19-22. Convention in New York City. Secy., P. J. McAndrews.

MICHIGAN GAS ASSOCIATION.
Sept. 21-22. Annual meeting in Detroit, Mich. Secy., Clark R. Graves, Lansing, Mich.

AMERICAN PEAT SOCIETY.
Sept. 21-23. Annual meeting in Washington, D. C. Secy., Julius Bordollo, Kingsbridge, N. Y.

RAILWAY FIRE PROTECTION ASSOCIATION.
Oct. 3-5. Convention in New York City. Secy., C. B. Edwards, Mobile & Ohio R.R., Mobile, Ala.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 9-13. Convention in Newark, N. J. Secy., Charles C. Brown, Indianapolis, Ind.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 9-13. Convention at Atlantic City, N. J. Secy., E. B. Burritt, 8 West 40th St., New York City.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 17-19. Convention in New Orleans. Secy., C. A. Lichty, C. & N. W. Ry., Chicago.

AMERICAN GAS INSTITUTE.
Oct. 17-20. Annual meeting in Chicago. Secy., G. G. Ramsdell, New York, N. Y.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Oct. 24-27. Convention in Cincinnati, Ohio. Secy., Prof. Selskar M. Gunn, Boston, Mass.

The Traveling Engineers Association will hold its 24th annual convention in Chicago, at the Hotel Sherman, Sept. 5 to 8. Among the topics is: "Difficulties Accompanying the Prevention of Dense Black Smoke and Its Relation to Cost of Fuel and Locomotive Repairs." W. O. Thompson is secretary.

The Providence Engineering Society will occupy new rooms at 29 Waterman St., Providence, when it opens for its fall and winter seasons next September. The entire second floor of the building has been taken over and will be remodeled and equipped attractively. The secretary is Albert E. Thornley.

Society of Automobile Engineers—At the semi-annual meeting held in June definite proposals were made to change the name to Society of Automotive Engineers, in order to include engineers engaged in aëronautic, motor-boat, and similar work involving the use of internal-combustion engines. The society's Committee on Standards held a meeting

in July, with the National Advisory Committee for Aëronautics at the Bureau of Standards in Washington, at which plans were discussed for bringing about coöperation between the Federal government and designers and manufacturers of engines suitable for air craft.

Appliances and Materials

Valve for Dry-Pipe Sprinklers

A new model of dry-pipe valve for fire-protection sprinkler systems has been devised by the General Fire Extinguisher Co., Providence, R. I. This ("Model C") is the third type evolved in connection with the Grinnell automatic dry-pipe system for use where freezing temperatures prevent use of the ordinary or wet-pipe system. In the dry-pipe system compressed air is used to fill the pipes until a fire opens a sprinkler head; a special check valve is employed to hold back the water.

The new valve has a cast-iron body and a dished malleable hand-hole cover. The only essential moving part is a bronze clapper with a rubber air valve seating on babbitt and a bronze water valve and seat. The intermediate space between the two seats is a drain chamber vented to the air. The valve will admit water when the air pressure has dropped to ⅛ the water pressure. Unlike the earlier valves, the area of the air valve is only three times that of the water; the air-valve leverage is increased to compensate, giving as a net result a smaller and lighter valve.

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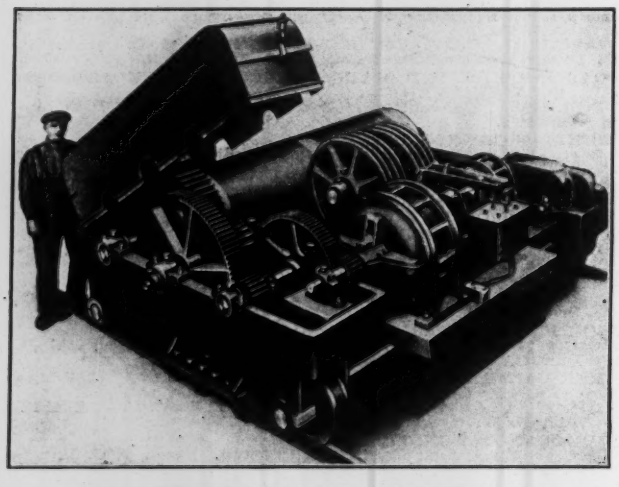
Large Alternating-Current Crane

One of the largest alternating-current cranes ever installed for handling locomotives has been furnished recently by the Whiting Foundry Equipment Co., Harvey, Ill., to the Seaboard Air Line Ry. for its new shops at Portsmouth, Va. The crane is of 160 tons capacity with two 80-ton trolleys, one of which is equipped with a 10-ton auxiliary hoist. The trolleys are of the construction shown in the accompanying view.

Among the features of the trolley may be noted the entire train of gears inclosed and run in oil; the motor pinion with outboard bearing; the idler-sheave mounting, which allows



EIGHTY-TON CRANE TROLLEY

the operator to inspect and oil while standing on top of the trolley and which increases the lift of the crane by about 30 in. With the exception of the drum shaft, no shafts extend across the trolley. All shafts in the gear train are on the same line and are of cast steel, machine cut. Pinions are forged, as is the drum gear. Trolley sides and separator are of cast steel, the housing of structural steel and provided with proper hand holes to allow inspection and easy removal in making repairs.

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Pump and Generator Outfits with Automobile Engines

An outfit recently introduced consists of an automobile-type gasoline engine and horizontal-shaft centrifugal pump, both mounted on a bedplate and having their shafts connected through a flywheel and clutch pulley. A connection to the pump supplies cooling water for the cylinders, but if the pump is handling dirty water a separate circulating pump or a circulating tank may be fitted. A four-cylinder engine of 7 hp. drives a pump with 4-in. suction and 3-in. discharge, and at a speed of 1,000 r.p.m. the outfit can deliver 260 gal. per min. against 50 ft. total head. The same type of engine can be connected to a 110-volt direct-current generator of 3 kw., mounted on the end of the bedplate. This can be used for a 10-in. searchlight, to operate lights up to 3,000 watts, or to drive a 4-hp. motor. These outfits are built by the Zin-Ho Manufacturing Co., of Chicago.

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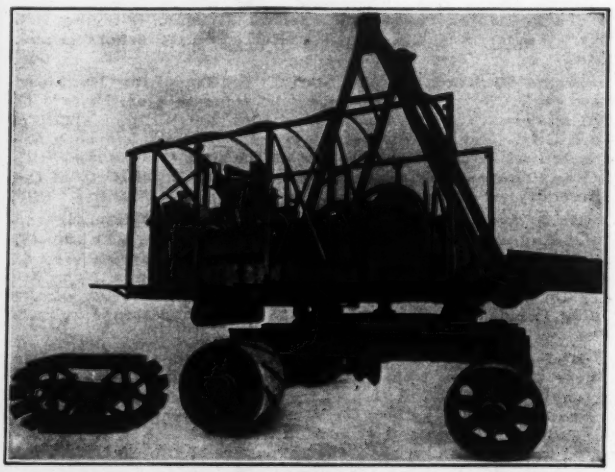
New Dragline Excavator

The traveling dragline excavator is being used very extensively and for a variety of purposes, and a new design of excavator of this type is shown in the accompanying view. It was designed by G. N. Crawford, excavator expert for the Pawling & Harnischfeger Co., of Milwaukee, Wis., and is built by that company.

In this machine the revolving frame has as its main element a pair of heavy plate girders, carrying the A-frame, boom and all machinery. It has a pivot and center casting of annealed steel, and is supported by rollers traveling on a circular track of 60-lb. T-rail riveted upon the lower frame. This frame is built up of I-beams and plates. The eight rollers are arranged in pairs, each pair being mounted in a truck frame. A pinion on a vertical shaft gears with a circular rack inside the roller path. The boom is composed of a pair of steel channels, with top and bottom bracing, and is heavily reinforced at the end for the head sheave and the



FRAME CONSTRUCTION AND MACHINERY OF CRAWFORD EXCAVATOR

eye-bar links carrying the crosshead to which the topping-lift block is attached.

The excavator is shown equipped with a multicylinder gasoline engine, but it can be fitted with a steam engine or electric motor, the current for the latter being taken from an outside source or generated on the machine. There are three drums, operating the dragline, the hoisting line and the topping-lift for the boom. The band frictions of the drums are operated by compressed-air rams. The equipment includes a tubular water-cooling radiator with fan, an air compressor, a centrifugal circulating pump, a gasoline tank and a receiver for compressed air. The tank and receiver are placed beneath the revolving frame. The gears are of annealed cast steel and all spur gears are cut from solid steel blanks. Water circulation and force-feed lubrication are effected automatically.

The machine can be mounted either on wide-tired wheels or on four caterpillars known as corduroy grips. Power is supplied from the engine through bevel gears to the rear shaft. The caterpillars are driven from the live axle by internal steel sprockets and chain drive from the rear axle. The wheels are of cast steel and have three rows of spokes. The rear or driving axle is built up of 12-in. I-beams, with stub ends which admit of renewal in case of wear. The axles are so arranged as to give a three-point support to the upper frame when traveling, but when the excavator is working, this frame is held rigidly in position by means of jacknuts.

The excavator illustrated has a 40-ft. boom handling a 1-yd. bucket, and is operated by a four-cylinder vertical gasoline engine of 70 hp. The turntable track is 8 ft. in diameter and the machine can be swung at the rate of 2½ r.p.m. The weight in working order is 30 tons.